CHAPTER 12

STORAGE FACILITIES

CHAPTER 12 – TABLE OF CONTENTS

12.1 INTRODUCTION

12.1.1 Overview

The traditional design of storm drainage systems has been to collect and convey storm runoff as rapidly as possible to a suitable location where it can be discharged. As areas urbanize, this type of design may result in major drainage and flooding problems downstream. The engineering community is now more conscious of the quality of the environment and the impact that uncontrolled increases in runoff can have on our customers. Under favorable conditions, the temporary storage of some of the storm runoff can decrease downstream flows and often the cost of the downstream conveyance system. Detention storage facilities can range from small facilities contained in parking lots or other on-site facilities to large lakes and reservoirs. This Chapter provides general design criteria for detention/retention storage basins and procedures for performing preliminary and final sizing and reservoir routing calculations. Storage and flood routing associated with culverts is addressed in the Culverts Chapter*. Note: Criteria in this Chapter do not necessarily apply to routine culvert design*.

12.1.2 Location Considerations

The location of storage facilities is very important because it relates to the effectiveness of these facilities to control downstream flooding. Small facilities will only have minimal flood control benefits, and these benefits will quickly diminish as the flood wave travels downstream. Multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the conveyance system, which could decrease or increase flood peaks in different downstream locations. Thus, it is important for the designer to design storage facilities as a drainage structure that both controls runoff from a defined area and interacts with other drainage structures within the drainage basin. Effective stormwater management must be coordinated on a regional or basin-wide planning basis. The Department should encourage and participate in such planning.

It is important to recognize the present and future land value when planning a location for storm drainage detention/retention. To all extent practicable, retention/detention basins should be placed in present and/or future low value land, possibly where the standing water that may be detain in the ponds not become a nuisance to the nearby developments.

12.1.3 Detention and Retention

Urban stormwater storage facilities are often referred to as either detention or retention facilities. For this Chapter, detention facilities are those that are designed to reduce the peak discharge and only detain runoff for some short period of time. These facilities are designed to completely drain after the design storm has passed. Recharge basins are a special type of detention basin designed to drain into the groundwater table; these are not addressed in this *Manual*. Retention facilities are designed to contain a permanent pool of water. Because most of the design procedures are the same for detention and retention facilities, the term storage facilities will be used in this Chapter to include detention and retention facilities. If special procedures are needed for detention or retention facilities, these will be specified.

Storage facilities may be small in terms of storage capacity and dam height where serving a single outfall from a watershed of a few hectares, or they may be larger facilities serving as regional stormwater management control. Although the same principles apply to all storage facilities, Sections 12.12 and 12.13 more specifically relate to the smaller installations.

12.1.4 Computer Programs

Routing calculations needed to design storage facilities, although not extremely complex, are time consuming and repetitive. To assist with these calculations, there are many available reservoir routing computer programs. Also, the storage indication method can be used which makes calculations simple. All storage facilities which will significantly affect peak outlet flows shall be designed and analyzed using reservoir routing calculations.

12.2 USES

12.2.1 Introduction

The use of storage facilities for stormwater management has increased dramatically in recent years. The benefits of storage facilities can be divided into two major control categories of quality and quantity.

12.2.2 Quality

Control of stormwater quality using storage facilities offers the following potential benefits:

- decrease downstream channel erosion;
- control sediment deposition; and
- improve water quality through:
    - stormwater filtration, and
    - capture of the first flush with detention for 24 h or more.

12.2.3 Quantity

Controlling the quantity of stormwater using storage facilities can provide the following potential benefits:

- prevention or reduction of peak runoff rate increases caused by urban development,
- mitigation of downstream drainage capacity problems,
- recharge of groundwater resources,
- reduction or elimination of the need for downstream outfall improvements, and
- maintenance of historic low-flow rates by controlled discharge from storage.

12.2.4 Objectives

The objectives for managing stormwater quantity by storage facilities are typically based on limiting peak runoff rates to match one or more of the following values:

- historic rates for specific design conditions (i.e., post-development peak equals pre-development peak for a particular frequency of occurrence);

- non-hazardous discharge capacity of the downstream drainage system; and

- a specified value for allowable discharge set by a regulatory jurisdiction.

For a watershed without an adequate outfall, the total volume of runoff is critical and retention storage facilities are used to store the increases in volume and to control discharge rates.

12.3 SYMBOLS AND DEFINITIONS

To provide consistency within this Chapter and throughout this *Manual*, the symbols in Table 12-1 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this Chapter, the symbol will be defined in the text or equations.

TABLE 12-1 — Symbols and Definitions

Symbol	Definition	Units
A	Cross sectional or surface area	ft^2
C	Weir coefficient	—
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
f	Infiltration rate	in/h
g	Acceleration due to gravity	ft/s^2
H	Head on structure	ft
H_c	Height of weir crest above channel bottom	ft
I	Infiltration rate	in/h
I	Inflow rate	ft^3/s
L	Length	ft
Q, O	Flow or outflow rate	ft^3/s
S_a	Surface area	ac
S, V_s	Storage volume	ft^3, ac•ft
t	Routing time period	s
t_b	Time base on hydrograph	h
T_i	Duration of basin inflow	h
t_p	Time to peak	h
W	Width of basin	ft
z	Side slope factor	—

12.4 POLICY

If the storm water runoff exceeds 5 cfs, obtain a Storm Water Discharge Permit from the Utah Department of Environmental Quality (UDEQ) for any new construction of a storm sewer to comply with the requirements of the Utah Division of Water Quality (UDWQ). Submit complete plans for a storm sewer to the Region Hydraulic Engineer for review, and then send to the Utah State DWQ for approval and issuance of a permit.

Retention or detention facilities designed to hold 20acre-ft or more shall be approved by UDOT Central Hydraulics.

12.5 DESIGN CRITERIA

12.5.1 General Criteria

Storage may be concentrated in large basin-wide or regional facilities or distributed throughout an urban drainage system. Storage may be developed in depressed areas in parking lots, road embankments and freeway interchanges, parks and other recreational areas and small lakes, ponds and depressions within urban developments. The utility of any storage facility depends on the amount of storage, its location within the system and its operational characteristics. An analysis of such storage facilities should consist of comparing the design flow at a point or points downstream of the proposed storage site with and without storage. In addition to the design flow, other flows in excess of the design flow that might be expected to pass through the storage facility should be included in the analysis. The design criteria for storage facilities should include:

- release rate,
- storage volume,
- grading and depth requirements,
- outlet works and location, and
- provisions for maintenance (such as berms and access ramps).

12.5.2 Release Rate

Design discharge from the detention facilities shall approximate pre-developed peak runoff rates of no more than 0.2 cfs per acre. Design calculations are required to demonstrate no more than 0.2 cfs per acre from the service area will be discharged when the detention facility is at 50% capacity. Multi-stage control structures may be required to control both runoff from the 2- and 10-yr storms.

12.5.3 Storage

Storage volume shall be adequate to attenuate the post-development peak discharge rates to pre-developed discharge rates. In urban or urbanizing areas, storage volume should be sufficient for the 10-yr storm, or for the storm drain system capacity, whichever is greater. Local entities (city or counties) may have different requirements for designing storage facilities. The designer should coordinate with local entities and try to accommodate their drainage master plans when designing storage facilities.

In rural areas, the design storm may vary, depending on the downstream system capacity. Routing calculations must be used to show that the storage volume is adequate. If sedimentation during construction causes loss of detention volume, design dimensions shall be restored before completion of the project. The designer shall provide means by which all detention volume be drained to provide maintenance for the facility. It is suggested that the designer evaluate erodability of disturbed areas tributary to the pond and if necessary provide for initial maintenance at the completion of the project to restore the full sediment storage capacity.

12.5.4 Grading and Depth

Following is a discussion of the general grading and depth criteria for storage facilities followed by criteria related to detention and retention facilities.

12.5.4.1 General

The construction of storage facilities usually requires excavation or placement of earthen embankments to obtain sufficient storage volume. Vegetated embankments shall be less than 10 ft in height and shall have side slopes no steeper than 1V:3H. Side slopes shall be benched at intervals of 5 ft. Riprap-protected embankments shall be no steeper than 1V:2H. Geotechnical slope stability analysis is recommended for embankments greater than 10 ft in height and is mandatory for embankment slopes steeper than those given above. Procedures for performing slope stability evaluations can be found in most soil engineering textbooks, including those by Spangler and Handy*(13)* and Sowers and Sowers *(12)*.

A minimum freeboard of 1 ft above the 100-yr design storm high-water elevation shall be provided for impoundment depths of less than 20 ft. Impoundment depths greater than 20 ft or volumes greater than 30 ac•ft are subject to the requirements of the Safe Dams Act (see Section 12.6), unless the facility is excavated to this depth.

Other considerations when setting depths include flood elevation requirements, public safety, land availability, land value, present and future land use, water table fluctuations, soil characteristics, maintenance requirements and required freeboard. Aesthetically pleasing features are also important in urbanizing areas. Fencing of basins is addressed in Section 12.19.

12.5.4.2 Detention

Areas above the normal high-water elevations of storage facilities should be sloped at a minimum of 5% toward the facilities to allow drainage and to prevent standing water. Careful finish grading is required to avoid creation of upland surface depressions that may retain runoff. The bottom area of storage facilities should be graded toward the outlet to prevent standing water conditions. A minimum 2% bottom slope is recommended. A low-flow or pilot channel constructed across the facility bottom from the inlet to the outlet is recommended to convey low flows and prevent standing water conditions.

12.5.4.3 Retention

The maximum depth of permanent storage facilities will be determined by site conditions, design constraints and environmental needs. In general, if the facility provides a permanent pool of water, a depth sufficient to discourage growth of weeds (without creating undue potential for anaerobic bottom conditions) should be considered. A depth of 5 ft to 10 ft is generally reasonable unless safety or fishery or requirements dictate otherwise. The location should be fenced if it is in an urban area and it is accessible to the public. Aeration may be required in permanent pools to prevent anaerobic conditions. Where aquatic habitat is required, the appropriate wildlife experts should be contacted for site-specific criteria relating to such elements as depth, habitat, and bottom and shore geometry.

12.5.5 Outlet Works

Outlet works selected for storage facilities typically include a principal spillway and an emergency overflow and must be able to accomplish the design functions of the facility. Outlet works can take the form of combinations of drop inlets, pipes, weirs and orifices. Curb openings may be used for parking lot storage. The principal spillway is intended to convey the design storm without allowing flow to enter an emergency outlet. For large storage facilities, selecting a flood magnitude for sizing the emergency outlet should be consistent with the potential threat to downstream life and property if the basin embankment were to fail. The minimum flood to be used to size the emergency outlet is the 100-yr flood. The sizing of a particular outlet works shall be based on the results of hydrologic routing calculations.

12.5.6 Location

In addition to controlling the peak discharge from the outlet works, storage facilities will change the timing of the entire hydrograph. If several storage facilities are located within a particular basin, it is important to determine what effects a particular facility may have on combined hydrographs in downstream locations. For all storage facilities, ((channel routing calculations shall proceed downstream to a confluence point where the drainage area being analyzed represents 10% of the total drainage area. At this point, the effect of the hydrograph routed through the proposed storage facility on the downstream hydrograph shall be assessed for detrimental effects on downstream areas.

12.6 SAFE DAMS ACT

12.6.1 Background

National responsibility for the promotion and coordination of dam safety lies with FEMA. Responsibility for administrating laws and regulations pertaining to dam safety is given to the Utah Division of Water Rights. Rules and regulations relating to applicable dams are promulgated by the Utah Division of Water Resources.

Under the Federal regulations, a dam is an artificial barrier that does or may impound water that is 27 ft or greater in height or has a maximum storage volume of 30 ac•ft or more. A number of exemptions are allowed from the Safe Dams Act and the Utah Division of Water Resources should be contacted to resolve questions.

Any roadway project on or adjacent to an existing dam shall be evaluated and coordinated with the responsible owner.

12.6.2 Classification

Dams are classified as either new or existing, by hazard potential and by size. Hazard potential categories are listed below:

CATEGORY 1. Dams are located where failure would probably result in loss of human life, excessive economic loss due to damage of downstream properties, public hazard or public inconvenience due to loss of impoundment and/or damage to roads or any public or private utilities.

CATEGORY 2. Dams are located where failure may damage downstream private or public property, but such damage would be relatively minor and within the general financial capabilities of the dam owner. Public hazard or inconvenience due to loss of roads or any public or private utilities would be minor and of short duration. Chances of loss of human life would be possible but remote.

CATEGORY 3. Dams are located where failure may damage uninhabitable structures or land, but such damage would probably be confined to the dam owner's property. No loss of human life would be expected.

12.7 GENERAL PROCEDURE

12.7.1 Data Needs

The following data will be needed to complete storage design and routing calculations:

- inflow hydrograph for all selected design storms;
- stage-storage curve for proposed storage facility (see Figure 12-1 for an example). For large storage volumes (e.g., for reservoirs), use ac•ft; otherwise use cubic feet; and
- stage-discharge curve for all outlet control structures (see Figure 12-2 for an example).

Using these data, a design procedure is used to route the inflow hydrograph through the storage facility to establish an outflow hydrograph. If the desired outflow results are not achieved, basin and outlet geometry are varied to yield new stage-storage and stage-discharge curves and the routing procedure is redone until the desired outflow hydrograph is achieved (see Example 12.9).

12.7.2 Stage-Storage Curve

A stage-storage curve defines the relationship between the depth of water and storage volume in a reservoir. The data for this type of curve are usually developed using a topographic map and one of the following formulas — the average-end area, frustum of a pyramid or prismoidal formulas. Storage basins are often irregular in shape to blend well with the surrounding terrain and to improve aesthetics. Therefore, the average-end area formula is usually preferred as the method to be used on non-geometric areas. The average-end area formula is expressed as:

$$V_{1,2} = [(A_1 + A_2)/2]d \tag{12.1}$$

where: $V_{1,2}$ = storage volume, ft^3, between Elevations 1 and 2
$A_{1,2}$ = surface area at Elevations 1 and 2 respectively, ft^2
D = change in elevation between Points 1 and 2, ft

The frustum of a pyramid is expressed as:

$$V = d/3\ [A_1 + (A_1A_2)^{0.5} + A_2] \tag{12.2}$$

where: V = volume of frustum of a pyramid, ft^3
d = change in elevation between Points 1 and 2, ft
$A_{1,2}$ = surface area at Elevations 1 and 2 respectively, ft^2

The prismoidal formula for trapezoidal basins is expressed as:

$$V = LWD + (L + W)\ ZD^2 + 4/3\ Z^2\ D^3 \tag{12.3}$$

where: V = volume of trapezoidal basin, ft^3

L = length of basin at base, ft
W = width of basin at base, ft
D = depth of basin, ft
Z = side slope factor, ratio of vertical to horizontal

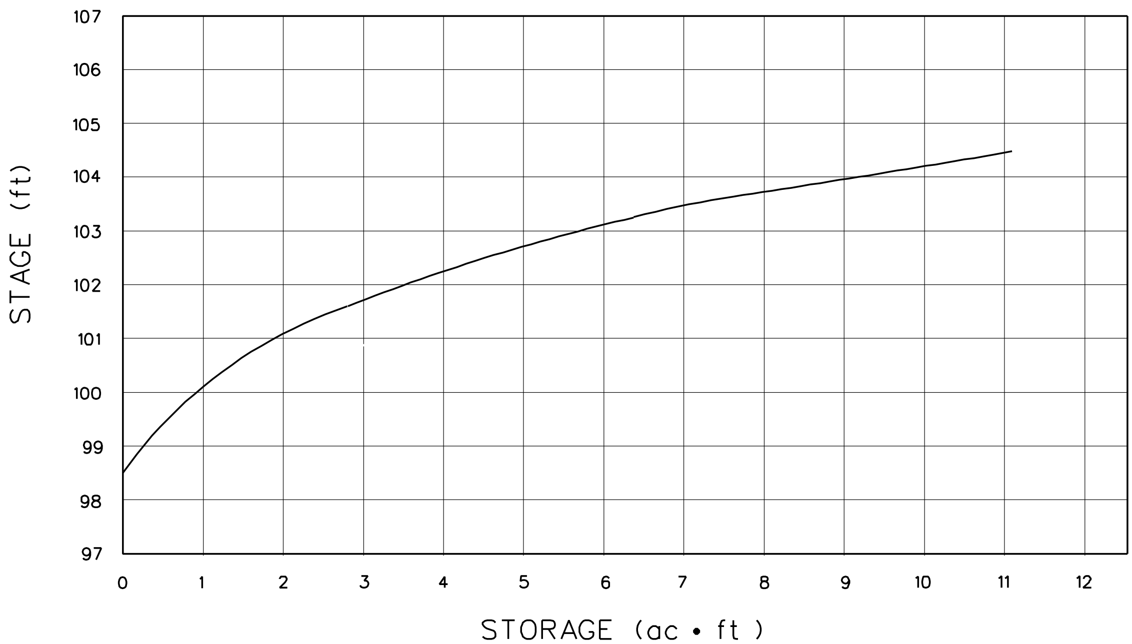


FIGURE 12-1 — Example Stage-Storage Curve

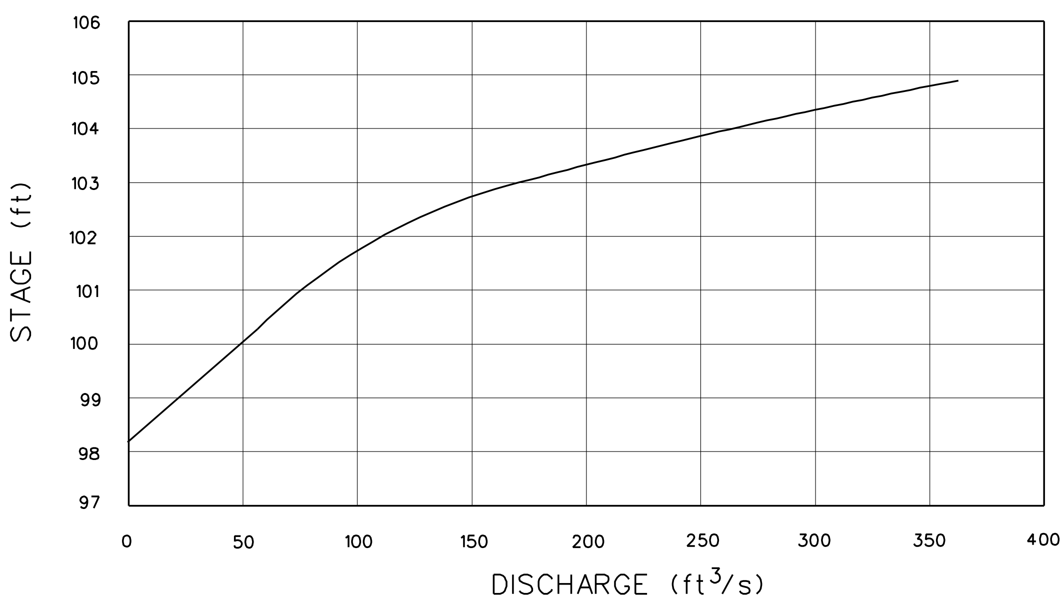


FIGURE 12-2 — Example Stage-Discharge Curve

It should be noted that Microstration with Inroads will automatically develop a stage-storage curve for the designed basin. Inroads interpolates from the existing ground surface to the new surface to compute volumes of excavation vs. elevation.

12.7.3 Stage-Discharge Curve

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility. A typical storage facility has two spillways — principal and emergency. The principal spillway is usually designed with a capacity sufficient to convey the design flood without allowing flow to enter the emergency spillway. A pipe culvert, weir or other appropriate outlet can be used for the principal spillway or outlet. Tailwater influences and structure losses must be considered when developing discharge curves.

The emergency spillway is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal spillway. This spillway should be designed considering the potential threat to downstream life and property if the storage facility were to fail.

The stage-discharge curve should reflect the discharge characteristics of both the principal and emergency spillways.

12.7.4 Procedure

A suggested procedure for using the above data in the design of storage facilities is presented below:

Step 1 Compute inflow hydrograph for runoff from the 2-, 10- and 100-yr design storms using the procedures outlined in the Hydrology Chapter. Only the post-development hydrograph is required for runoff from the 100-yr design storm.

Step 2 Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1 (see Section 12.9). If storage requirements are satisfied for runoff from the 2-, 10- and 100-yr design storms, runoff from intermediate storms is assumed to be controlled.

Step 3 Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. The maximum storage requirement calculated from Step 2 should be used.

Step 4 Size the outlet structure. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure should be sized to convey the allowable discharge at this stage.

Step 5 Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using the storage routing equations. If the routed post-development peak discharges from the 2-, 10- and 100-yr design storms exceed the pre-development peak discharges, or if the peak stage varies significantly from the estimated peak stage from Step 4, then revise the estimated volume and return to Step 3.

Step 6 Consider emergency overflow from runoff due to 100-yr or larger design storm and established freeboard requirements. The 100-year storm should routed to the ultimate outfall to identify potential flood hazards.

Step 7 Evaluate the downstream effects of detention outflow to ensure that the routed hydrograph does not cause downstream flooding problems. The exit hydrograph from the storage facility should be routed though the downstream channel system until a confluence point is reached where the there is no flooding risk.

Step 8 Evaluate the control structure outlet velocity and provide channel and bank stabilization if the velocity will cause erosion problems downstream.

This procedure can involve a significant number of reservoir routing calculations to obtain the desired results.

12.8 OUTLET HYDRAULICS

12.8.1 Outlets

Sharp-crested weir flow equations for no end contractions, two end contractions and submerged discharge conditions are presented below, followed by equations for broad-crested weirs, V-notch weirs, proportional weirs and orifices, or combinations of these facilities. If culverts are used as outlets works, procedures presented in the Culverts Chapter should be used to develop stage-discharge data. When analyzing release rates, the tailwater influence of the principal spillway culvert on the control structure (orifice and/or weirs) must be considered to determine the effective head on each opening.

12.8.2 Sharp-Crested Weirs

A sharp-crested weir with no end contractions is illustrated in Figure 12-3. The discharge equation for this configuration is *(3)*:

$$Q = [3.27 + 0.4(H/H_c)]\, LH^{1.5} \tag{12.4}$$

where: Q = discharge, ft^3/s
H = head above weir crest excluding velocity head, ft
H_c = height of weir crest above channel bottom, ft
L = horizontal weir length, ft

A sharp-crested weir with two end contractions is illustrated in Figures 12-4 and 12-5. The discharge equation for this configuration is *(3)*:

$$Q = [3.27 + 0.4(H/H_c)]\, (L - 0.2H)\, H^{1.5} \tag{12.5}$$

where: Variables are the same as Equation 12.4.

A sharp-crested weir will be affected by submergence where the tailwater rises above the weir-crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is *(1)*:

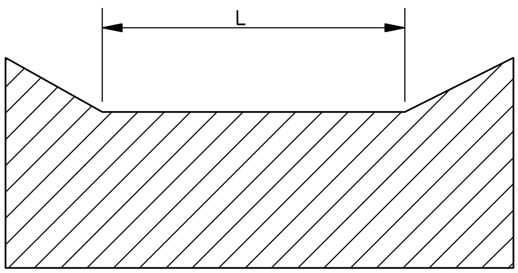


FIGURE 12-3 — Sharp-Crested Weir (No End Contractions)

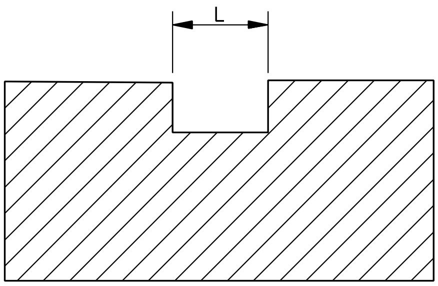


FIGURE 12-4 — Sharp-Crested Weir (Two End Contractions)

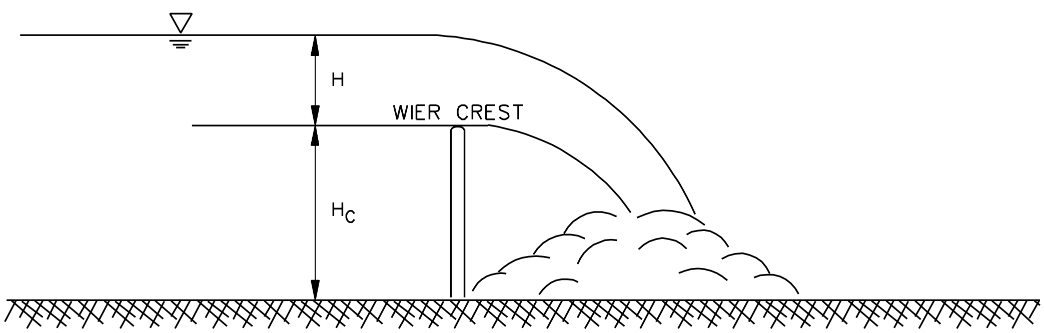


FIGURE 12-5 — Sharp-Crested Weir and Head

$$Q_S = Q_f(1 - (H_2/H_1)^{1.5})^{0.385} \quad (12.6)$$

where:
- Q_s = submergence flow, ft^3/s
- Q_f = free flow, ft^3/s
- H_1 = upstream head above crest, ft
- H_2 = downstream head above crest, ft

12.8.3 Broad-Crested Weirs

The equation generally used for the broad-crested weir is *(1)*:

$$Q = CLH^{1.5} \quad (12.7)$$

where:
- Q = discharge, ft^3/s
- C = broad-crested weir coefficient
- L = broad-crested weir length, ft
- H = head above weir crest, ft

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.087. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. Additional information on C values as a function of weir crest breadth and head is given in Table 12-2.

12.8.4 V-Notch Weirs

The discharge through a V-notch weir can be calculated from the following equation *(1)*:

$$Q = 2.5 \tan(\theta/2)H^{2.5} \quad (12.8)$$

where:
- Q = discharge, ft^3/s
- θ = angle of V-notch, degrees
- H = head on apex of notch, ft

12.8.5 Proportional Weirs

Although more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to vary nonlinearly with head.

Design equations for proportional weirs are *(10)*:

$$Q = 4.97\, a^{0.5}\, b(H - a/3) \quad (12.9)$$
$$x/b = 1 - (1/3.17)\,(\arctan\,(y/a)^{0.5}) \quad (12.10)$$

where:
- Q = discharge, ft^3/s

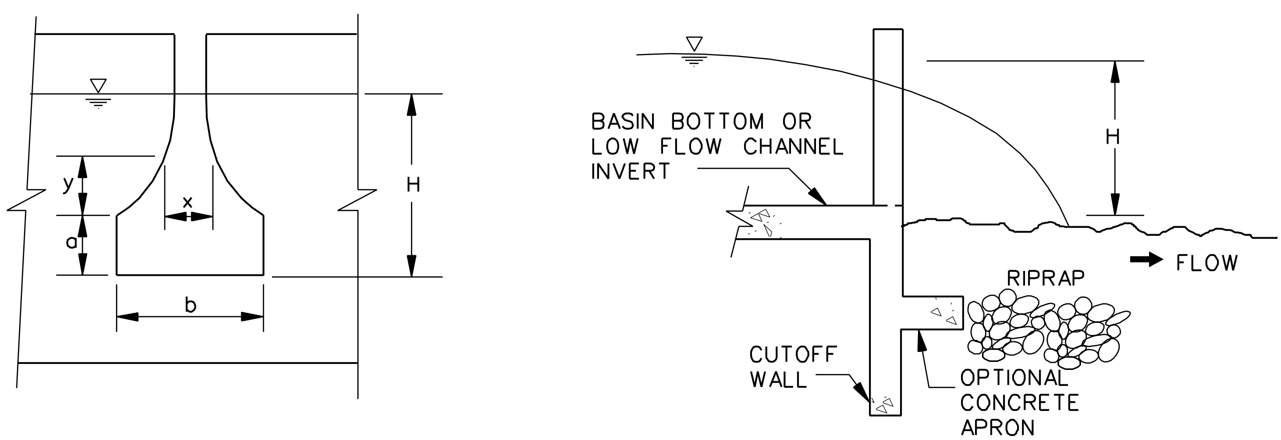
Dimensions a, b, H, x and y are shown in Figure 12-6.

TABLE 12-2 — Broad-Crested Weir Coefficient C Values as a Function of Weir Crest Breadth and Head (ft)

Measured Head, H[1] (ft)	Breadth of the Crest of Weir (ft)										
	0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.27	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

[1]Measured at least 2.5H upstream of the weir.

Source: Reference *(1)*.



Elevation

Section

FIGURE 12-6 — Proportional Weir Dimensions

12.8.6 Orifices

Pipes smaller than 12 in may be analyzed as a submerged orifice if H/D is greater than 1.5. For square-edged entrance conditions:

$$Q = 0.6A(2gH)^{0.5} \tag{12.11}$$

where:
- Q = discharge, ft^3/s
- A = cross-section area of pipe, ft^2
- g = acceleration due to gravity, 32.2 ft/s^2
- D = diameter of pipe, ft
- H = head on pipe, from the center of pipe to the water surface, ft *

* *Where the tailwater is higher than the center of the opening, the head is calculated as the difference in water surface elevations.*

12.9 PRELIMINARY DETENTION CALCULATIONS

12.9.1 Storage Volume

A preliminary estimate of the storage volume required for peak-flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with the standard triangular shapes shown in Figure 12-7.

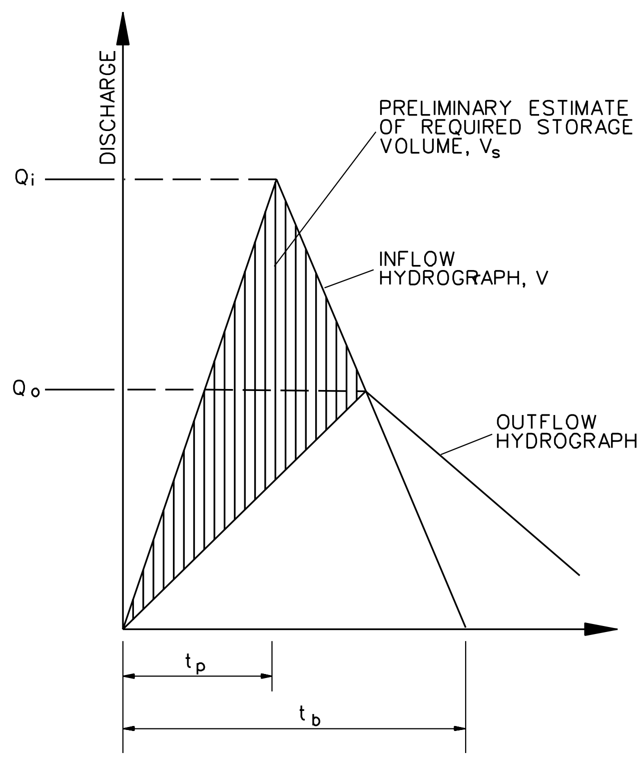


FIGURE 12-7 — Triangular Shaped Hydrographs
(For Preliminary Estimate of Required Storage Volume)

The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

$$V_S = 0.5T_i(Q_i - Q_o) \tag{12.12}$$

where:
- V_S = storage volume estimate, ft^3
- Q_i = peak inflow rate, ft^3/s
- Q_o = peak outflow rate, ft^3/s
- T_i = duration of basin inflow, s

Any consistent units may be used for Equation 12.12.

12.9.2 Alternative Method

An alternative preliminary estimate of the storage volume required for a specified peak-flow reduction can be obtained by the following regression equation procedure *(17)*:

1. Determine input data, including the allowable peak outflow rate, Q_o, the peak flow rate of the inflow hydrograph, Q_i, the time base of the inflow hydrograph, t_b, and the time to peak of the inflow hydrograph, t_p.

2. Calculate a preliminary estimate of the ratio V_S/V_r using the input data from Step 1 and the following equation:

$$V_S/V_r = [1.291(1 - Q_o/Q_i)^{0.753}]/[(t_b/t_p)^{0.411}] \tag{12.13}$$

   where:
   - V_S = volume of storage, ft^3
   - V_r = volume of runoff, ft^3
   - Q_o = outflow peak flow, ft^3/s
   - Q_i = inflow peak flow, ft^3/s
   - t_b = time base of the inflow hydrograph, h
     (Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5% of the peak).
   - t_p = time to peak of the inflow hydrograph, h

3. Multiply the peak-flow rate of the inflow hydrograph, Q_i, times the potential peak-flow reduction calculated in Step 2 to obtain the estimated peak outflow rate, Q_o, for the selected storage volume.

12.9.3 Peak-Flow Reduction

A preliminary estimate of the potential peak-flow reduction for a selected storage volume can be obtained by the following procedure:

1. Determine the following:

   - volume of runoff, V_r
   - peak-flow rate of the inflow hydrograph, Q_i
   - time base of the inflow hydrograph, t_b
   - time to peak of the inflow hydrograph, t_p
   - storage volume, V_S

2. Calculate a preliminary estimate of the potential peak-flow reduction for the selected storage volume using the following equation *(17)*:

$$Q_o/Q_i = 1 - 0.712(V_S/V_r)^{1.328}(t_b/t_p)^{0.546} \quad (12.14)$$

where:

Q_o = outflow peak flow, ft^3/s

Q_i = inflow peak flow, ft^3/s

V_S = volume of storage, ft^3

V_r = volume of runoff, ft^3

t_b = time base of the inflow hydrograph, h
(Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5% of the peak).

t_p = time to peak of the inflow hydrograph, h

3. Multiply the peak-flow rate of the inflow hydrograph, Q_i, times the potential peak-flow reduction calculated from Step 2 to obtain the estimated peak outflow rate, Q_o, for the selected storage volume (see Section 12.11.3 of the Example).

12.9.4 Preliminary Basin Dimensions

CADD programs will speed this process. There are several CADD programs available on the market that allow the designer to quickly size the basin with digital terrain models (dtm).

- Plot the control structure location on a contour map.

- Select a desired depth of ponding for the design storm.

- Divide the estimated storage volume needed by the desired depth to obtain the surface area required of the reservoir.

- Based on site conditions and contours, estimate the geometric shape(s) required to provide the estimated reservoir surface area.

12.10 ROUTING CALCULATIONS

The following procedure is used to perform routing through a reservoir or storage facility (Puls Method of storage routing *(14)*):

Step 1 Develop an inflow hydrograph, stage-discharge curve and stage-storage curve for the proposed storage facility. An example stage-storage curve is shown in Figure 12-8, and a stage-discharge curve is shown in Figure 12-9.

Step 2 Select a routing time period, Δt, to provide at least five points on the rising limb of the inflow hydrograph ($t < T_c/5$).

Step 3 Use the storage-discharge data from Step 1 to develop storage characteristics curves that provide values of $S \pm (O/2)\Delta t$ versus stage. An example tabulation of storage characteristics curve data is shown in Table 12-3.

Step 4 For a given time interval, I_1 and I_2 are known. Given the depth of storage or stage, H_1, at the beginning of that time interval, $S_1 - (O_1/2)\Delta t$ can be determined from the appropriate storage characteristics curve (Figure 12-10).

Step 5 Determine the value of $S_2 + (O_2/2)\Delta t$ from the following equation:

$$S_2 + (O_2/2)\Delta t = [S_1 - (O_1/2)\Delta t] + [(I_1 + I_2)/2]\Delta t \qquad (12.15)$$

where:
- S_2 = storage volume at Time 2, ft^3
- O_2 = outflow rate at Time 2, ft^3/s
- Δt = routing time period, s
- S_1 = storage volume at Time 1, ft^3
- O_1 = outflow rate at Time 1, ft^3/s
- I_1 = inflow rate at Time 1, ft^3/s
- I_2 = inflow rate at Time 2, ft^3/s

Other consistent units are equally appropriate.

Step 6 Enter the storage characteristics curve at the calculated value of $S_2 + (O_2/2)\Delta t$ determined in Step 5 and read a new depth of water, H_2.

Step 7 Determine the value of O_2, which corresponds to a stage of H_2 determined in Step 6, using the stage-discharge curve.

Step 8 Repeat Steps 1 through 7 by setting new values of I_1, O_1, S_1 and H_1 equal to the previous I_2, O_2, S_2 and H_2 and using a new I_2 value. This process is continued until the entire inflow hydrograph has been routed through the storage basin.

12.11 EXAMPLE PROBLEM

12.11.1 Example

This Example demonstrates the application of the methodology presented in this Chapter for the design of a typical detention storage facility. Example inflow hydrographs and associated peak discharges for both pre- and post-development conditions are assumed to have been developed using hydrologic methods from the Hydrology Chapter.

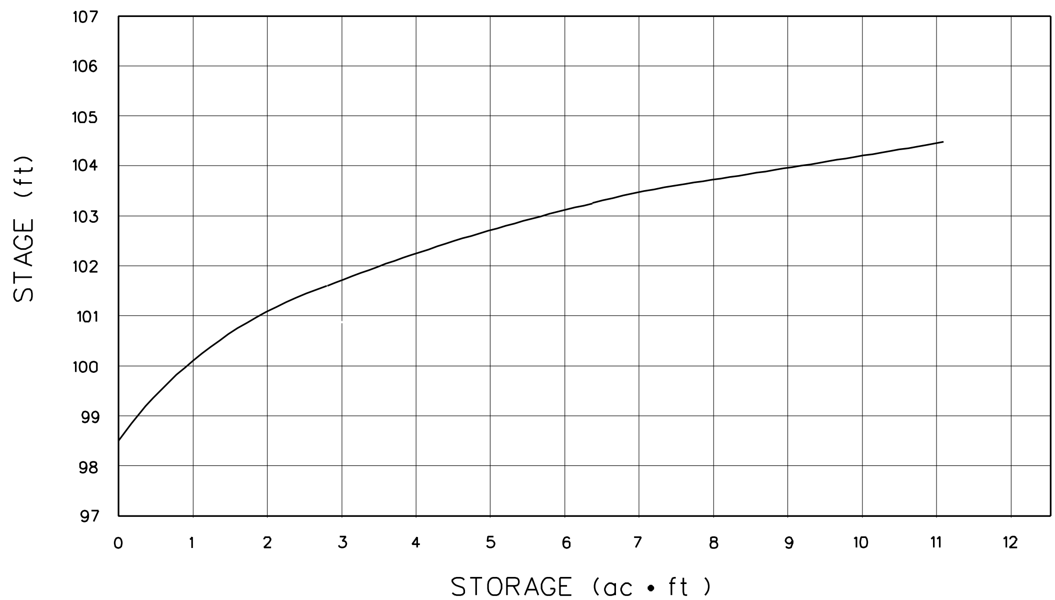


FIGURE 12-8 — Example Stage-Storage Curve

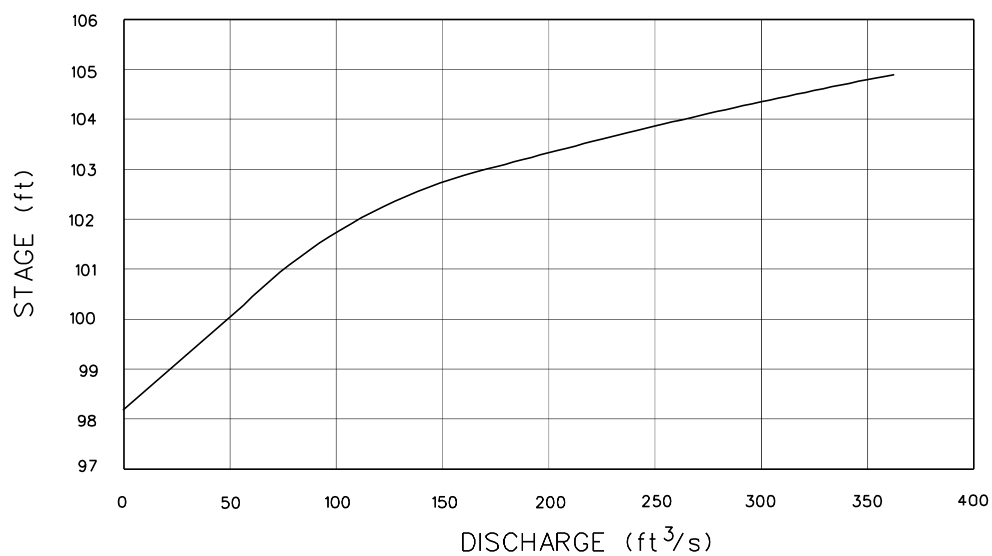


FIGURE 12-9 — Example Stage-Discharge Curve

TABLE 12-3 — Storage Characteristics

(1) Stage (ft)	(2) Storage[1] (ac•ft)	(3) Discharge[2] (ft^3/s)	(4) S – (O/2)Δt (ac•ft)	(5) S + (O/2)Δt (ft^3)
98.4	0.05	0	0.05	0.05
99.3	0.3	15	0.20	0.40
100.0	0.8	35	0.56	1.04
100.9	1.6	63	1.17	2.03
101.7	2.8	95	2.15	3.45
102.5	4.4	143	3.41	5.39
103.4	6.6	200	5.22	7.98
104.2	10.0	275	8.11	11.89

[1] *Obtained from the Stage-Storage Curve.*
[2] *Obtained from the Stage-Discharge Curve.*

Note: t = 10 min = 0.167 h

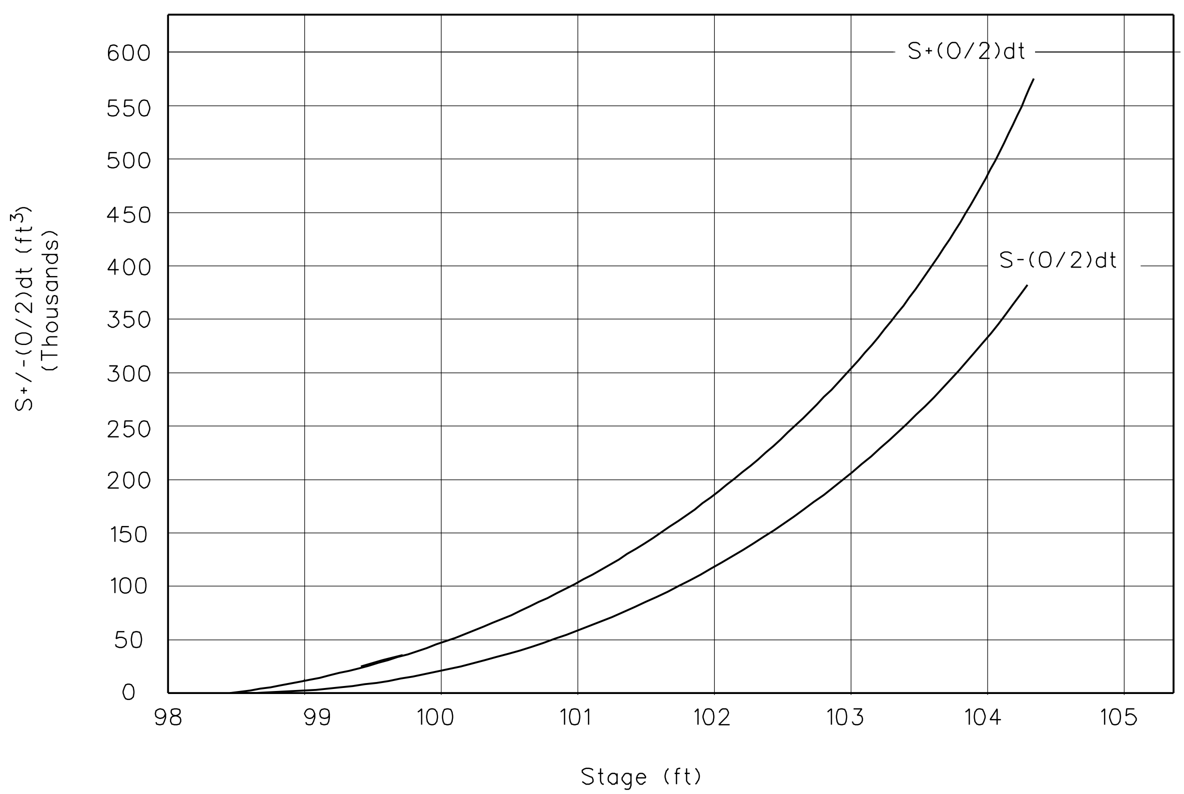


FIGURE 12-10 — Storage Characteristics Curve

12.11.2 Design Discharge and Hydrographs

Storage facilities are to be designed for runoff from both the 2- and 10-yr design storms and an analysis done using the 100-yr design storm runoff to ensure that the structure can accommodate runoff from this storm without damaging adjacent and downstream property and structures. Example peak discharges from the 2- and 10-yr design storm events are as follows:

- predevelopment 2-yr peak discharge = 150 ft^3/s
- predevelopment 10-yr peak discharge = 200 ft^3/s
- post-development 2-yr peak discharge = 190 ft^3/s
- post-development 10-yr peak discharge = 250 ft^3/s

Because the post-development peak discharge must not exceed the pre-development peak discharge, the allowable design discharges are 150 and 200 ft^3/s for the 2- and 10-yr storms, respectively.

Example runoff hydrographs are shown in Table 12-4. Inflow durations from the post-development hydrographs are approximately 1.2 and 1.25 h, respectively, for runoff from the 2- and 10-yr storms.

12.11.3 Preliminary Volume Calculations

Preliminary estimates of required storage volumes are obtained using the simplified method outlined in Section 12.9. For runoff from the 2- and 10-yr storms, the required storage volumes, V_S, are computed using Equation 12.12:

$$V_S = 0.5T_i(Q_i - Q_o)$$

2-yr storm: $V_S = [0.5(1.2)(3600)(190 - 150)]/43{,}560 = 1.98$ ac•ft

10-yr storm: $V_S = [0.5(1.25)(3600)(250 - 200)]/43{,}560 = 2.58$ ac•ft

TABLE 12-4 — Example Runoff Hydrographs

Predevelopment Runoff			Post-Development Runoff	
(1)	(2)	(3)	(4)	(5)
Time (h)	2-yr (ft^3/s)	10-yr (ft^3/s)	2-yr (ft^3/s)	10-yr (ft^3/s)
0.0	0	0	0	0
0.1	18	24	38	50
0.2	61	81	125	178
0.3	127	170	190 > 150	250 > 200
0.4	150	200	125	165
0.5	112	150	70	90
0.6	71	95	39	50
0.7	45	61	22	29
0.8	30	40	12	16
0.9	21	28	7	9
1.0	13	18	4	5
1.1	10	15	2	3
1.2	8	13	0	1

12.11.4 Design and Routing Calculations

Stage-discharge and stage-storage characteristics of a storage facility are shown in Table 12-5 that should provide adequate peak-flow attenuation for runoff from both the 2- and 10-yr design storms, which are shown in Tables 12-7 and 12-8. The storage-discharge relationship was developed by requiring the preliminary storage volume estimates of runoff for both the 2- and 10-yr design storms to be provided when the corresponding allowable peak discharges occurred. Discharge values were computed by solving the broad-crested weir equation for head, H, assuming a constant discharge coefficient of 3.1, a weir length of 4 ft and no tailwater submergence. The capacity of storage relief structures was assumed negligible.

Storage routing was conducted for runoff from both the 2- and 10-yr design storms to confirm the preliminary storage volume estimates and to establish design water surface elevations. Routing results using the Stage-Discharge-Storage data given previously and the Storage Characteristics Curves given on Figures 12-8 and 12-9, and 0.1-h time steps are shown below for runoff from the 2- and 10-yr design storms, respectively. The preliminary design provides adequate peak discharge attenuation for both the 2- and 10-yr design storms.

For the routing calculations, the following equation was used:

$$S_2 + (O_2/2)\Delta t = [S_1 - (O_1/2)\Delta t] + [(I_1 + I_2)/2\Delta t]$$

Also, Column 6 = Column 3 + Column 5.

Because the routed peak discharge is lower than the maximum allowable peak discharges for both design storm events, the weir length could be increased or the storage decreased. If revisions are desired, routing calculations must be repeated.

Although not shown for this Example, runoff from the 100-yr storm should be routed through the storage facility to establish freeboard requirements and to evaluate emergency overflow and stability requirements. In addition, the preliminary design provides hydraulic details only. Final design should consider site constraints (e.g., depth to water, side slope stability, maintenance, grading) to prevent standing water and provisions for public safety.

12.11.5 Downstream Effects

An estimate of the potential downstream effects (i.e., increased peak-flow rate and recession time) of detention storage facilities may be obtained by comparing hydrograph recession limbs from the pre-development and routed post-development runoff hydrographs. Example comparisons are shown below for the 10-yr design storms.

Potential effects on downstream facilities should be minor when the maximum difference between the recession limbs of the pre-developed and routed outflow hydrographs is less than approximately 20%. As shown in Figure 12-11, the example results are well below 20%; downstream effects can thus be considered negligible and downstream flood routing omitted. However, it is important to be aware that the increased total volumes of water being released slowly over a longer period of time may contribute to bed and bank decay in the receiving channel.

TABLE 12-5 — Stage-Discharge-Storage Data

(1)	(2)	(3)	(4)	(5)
Stage (ft)	Q (ft^3/s)	(ac•ft)	(ac•ft)	(ac•ft)
0.0	0	0.00	0.00	0.00
0.9	10	0.26	0.22	0.89
1.4	20	0.42	0.33	1.91
1.8	30	0.56	0.43	2.65
2.2	40	0.69	0.53	3.24
2.5	50	0.81	0.61	4.12
2.9	60	0.93	0.68	5.00
3.2	70	1.05	0.76	5.88
3.5	80	1.17	0.84	6.75
3.7	90	1.28	0.89	7.78
4.0	100	1.40	0.98	8.51
4.5	120	1.63	1.13	9.98
4.8	130	1.75	1.21	10.86
5.0	140	1.87	1.29	11.74
5.3	150	1.98	1.36	12.47
5.5	160	2.10	1.43	13.50
5.7	170	2.22	1.51	14.38
6.0	180	2.34	1.59	14.96
6.4	200	2.58	1.76	16.61
6.8	220	2.83	1.92	18.19
7.0	230	2.95	2.00	19.07
7.4	250	3.21	2.19	20.54

TABLE 12-6 — Storage Routing for the 2-Yr Storm

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Time (h)	Inflow (ft^3/s)	$[(I_1 + I_2)]/2$ (ac•ft)	H_1 (ft)	$S_1 - (O_1/2)\Delta t$ (ac•ft) (6) – (8)	$S_2 + (O_2/2)\Delta t$ (ac•ft) (3) + (5)	H_2 (ft)	Outflow (ft^3/s)
0.0	0	0.00	0.00	0.00	0.00	0.00	0
0.1	38	0.16	0.00	0.00	0.16	0.43	3
0.2	125	0.67	0.43	0.10	0.77	2.03	36
0.3	190	1.30	2.03	0.50	1.80	4.00	99
0.4	125	1.30	4.00	0.99	2.29	4.80	130<150 OK
0.5	70	0.81	4.80	1.21	2.02	4.40	114
0.6	39	0.45	4.40	1.12	1.57	3.60	85
0.7	22	0.25	3.60	0.87	1.12	2.70	55
0.8	12	0.14	2.70	0.65	0.79	2.02	37
0.9	7	0.08	2.08	0.50	0.58	1.70	27
1.0	4	0.05	1.70	0.42	0.47	1.03	18
1.1	2	0.02	1.30	0.32	0.34	1.00	12
1.2	0	0.01	1.00	0.25	0.26	0.70	7
1.3	0	0.00	0.70	0.15	0.15	0.40	3

TABLE 12-7 — Storage Routing for the 10-Yr Storm

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Time (h)	Inflow (ft^3/s)	$[(I_1 + I_2)]/2$ (ac•ft)	H_1 (ft)	$S_1 - (O_1/2)\Delta t$ (ac•ft (6) – (8)	$S_2 + (O_2/2)\Delta t$ (ac•ft) (3) + (5)	H_2 (ft)	Outflow (ft^3/s)
0.0	0	0.00	0.00	0.00	0.00	0.00	0
0.1	50	0.21	0.21	0.00	0.21	0.40	3
0.2	178	0.94	0.40	0.08	1.02	2.50	49
0.3	250	1.77	2.50	0.60	2.37	4.90	134
0.4	165	1.71	4.90	1.26	2.97	2.97	173 < 200 OK
0.5	90	1.05	5.80	1.30	2.35	4.00	137
0.6	50	0.58	4.95	1.25	1.83	4.10	103
0.7	29	0.33	4.10	1.00	1.33	3.10	68
0.8	16	0.19	3.10	0.75	0.94	2.40	46
0.9	9	0.10	2.40	0.59	0.69	1.90	32
1.0	5	0.06	1.90	0.44	0.50	1.40	21
1.1	3	0.03	1.40	0.33	0.36	1.20	16
1.2	1	0.02	1.20	0.28	0.30	0.90	11
1.3	0	0.00	0.90	0.22	0.22	0.60	6

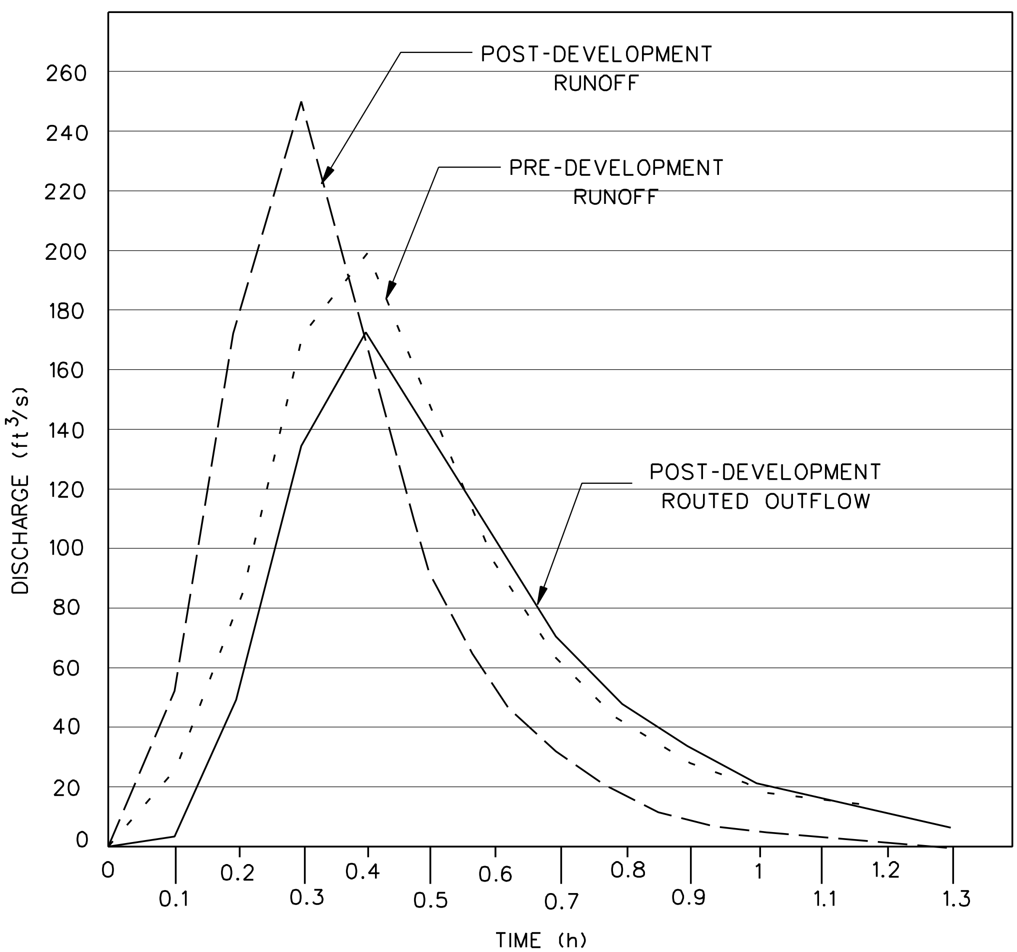


FIGURE 12-11— Runoff Hydrographs

12.12 WET POND (EXTENDED DETENTION BASIN)

12.12.1 Introduction

A wet pond is very similar to a dry detention basin in that it detains stormwater, but it is different in that it maintains a permanent pool during dry weather. Wet ponds are usually more expensive than dry detention basins and usually serve large watersheds. Because of their permanent pool, they may also have recreational benefits.

12.12.2 Design

12.12.2.1 Quality

In designing wet ponds for quality, it is suggested that the permanent pool approximately 3 times the WQV for the watershed. The theory behind this is that incoming runoff displaces old stormwater from the pond and the new runoff is detained until it is displaced by more runoff from the next storm. A permanent pool of 3 times the WQV should then provide an adequate detention time for the stormwater. Watershed size, soil conditions and groundwater elevation can be evaluated to ensure the capability of the site to support a permanent wet pond. Table 12-8 shows the permanent pool volume for different areas. To enhance pollutant removal, several other considerations can be considered.

The shape of the basin can significantly affect the pollutant-removal efficiency of a wet pond. The length-to-width ratio should be at least 3:1. Figure 12-12 shows pond configurations that may be used to increase the length-to-width ratio and allow for maximum flow path length. Pond depth should be between 4 ft to 5 ft; less could allow insect breeding and wind re-suspension of settled particles and more could lead to thermal stratification in the pond and anaerobic conditions in the deep water. A wedge-shaped basin, wider at the outlet, can also improve pollutant removal (see Figure 12-13).

TABLE 12-8 — Volumes and Flows According to Size of Area

Drainage Area (ac)	WQV (ft^3)	Average 30-h Outflow (ft^3/s)	Drainage Area (ac)	WQV (ft^3)	Average 30-h Outflow (ft^3/s)
2.5	4,590	0.04	27.2	50,493	0.47
4.9	9,181	0.08	30.0	56,496	0.51
7.4	13,771	0.13	32.1	59,674	0.55
9.9	18,361	0.17	34.6	64,264	0.60
12.4	22,952	0.21	37.1	68,855	0.64
14.8	27,542	0.25	39.5	73,445	0.68
17.3	32,132	0.30	42.0	78,035	0.72
19.8	36,722	0.34	44.5	82,625	0.77
22.2	41,313	0.38	46.9	87,216	0.81
24.7	45,903	0.42	49.4	91,806	0.85

12.12.2.2 Quantity

For quantity, the wet pond system is designed similarly to the dry pond. The pond should be designed to reduce the peak flow from a 2- and a 10-yr storm (considered individually) and be able to pass a 100-yr storm safely.

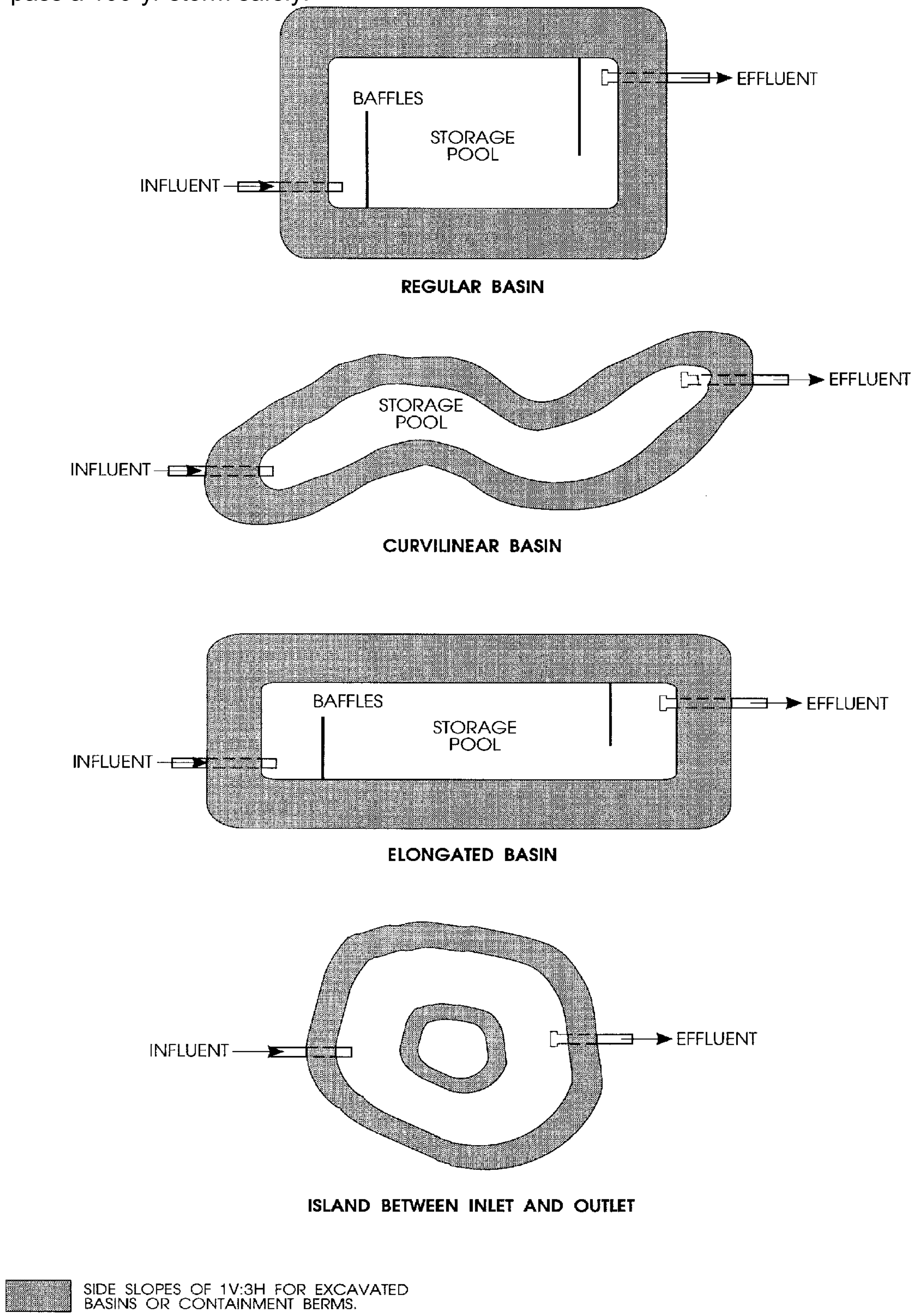


FIGURE 12-12 — Methods of Increasing the Length-to-Width Ratio (after Reference *(11)*)

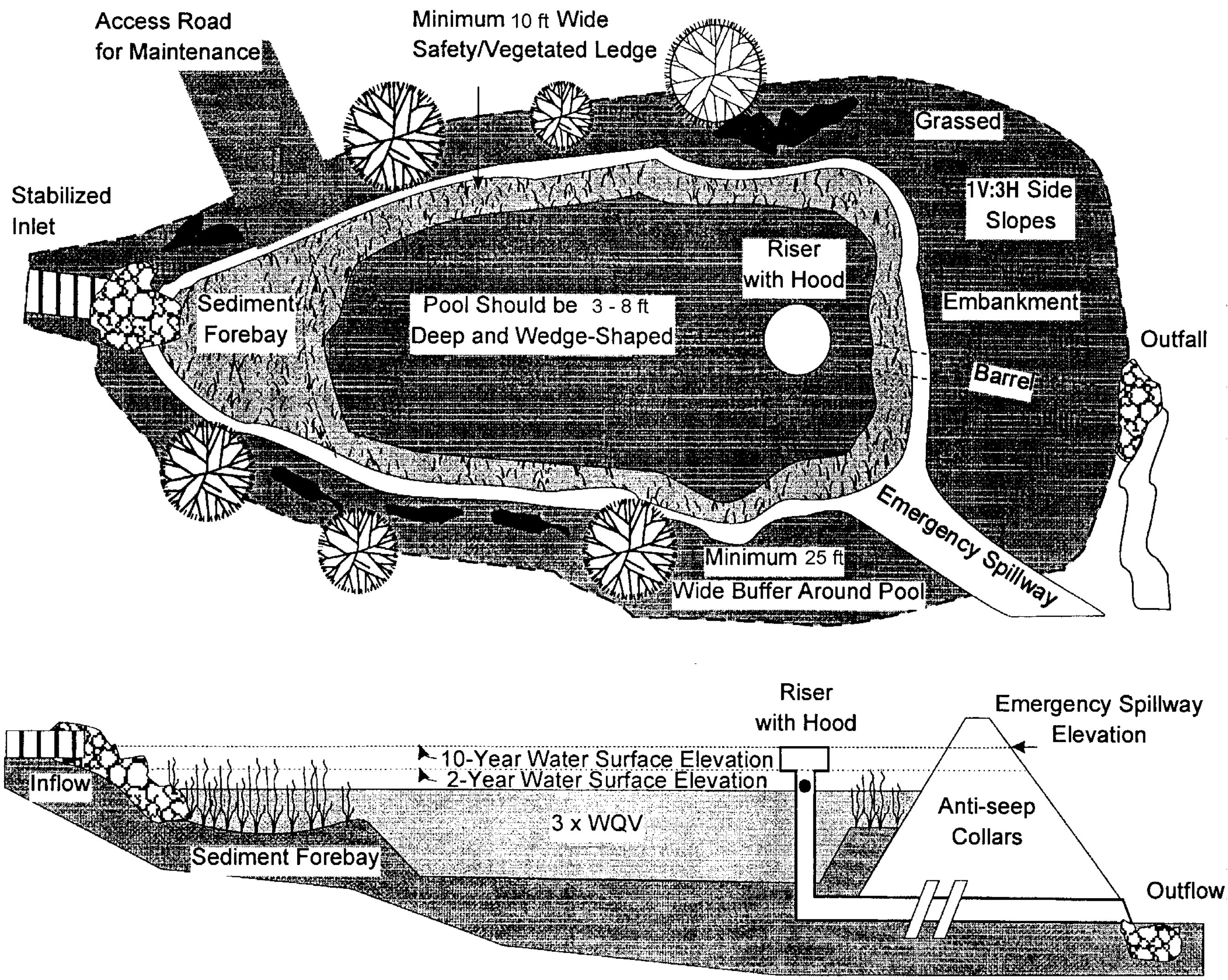


FIGURE 12-13 — Wet Pond (after Reference *(11)*)

Routing the storms through the wet pond can be accomplished in many ways.

12.12.2.3 Outlet

Outlets for wet ponds can be designed in a wide variety of configurations, but most outlets use riser pipes of concrete or corrugated metal. These risers can be designed to control different storms through the use of several orifices on the riser. For example, a small orifice is used to control a 2-yr storm and a larger orifice to control a 10-yr storm. This larger flow is usually controlled by allowing stormwater to flow in through the top of the riser, using the entire riser diameter. In some cases, an antivortex design may be necessary. Larger flood flows are usually handled by an emergency spillway. If the smallest orifice is easily clogged from floating debris, or if heated pond water causes problems downstream during the summer, the outlet can be modified so that it will release water from below the surface of the pond. Trash racks may also be included to prevent the outlet from clogging.

12.12.2.4 Summary

See Table 12-9.

TABLE 12-9 — Summary of Considerations for a Wet Pond

Quality	Permanent pool volume is 3 times the WQV.
Quantity	Control 2- and 10-yr peak flows.
Shape	3:1 length-to-width ratio; wedge shaped (wider at outlet); permanent pool depth from 5 ft – 10 ft; perimeter ledges.
Maintenance	Inspect once a year, preferably during wet weather; mow at least twice a year)); remove sediment every 5 to 10 years.
Safety	Fence around pond; provide shallow 1.5 ft deep safety ledge around pond; post signs.
Other considerations	Side slopes provide easy maintenance access 1V:3H; perimeter vegetation; sediment forebay; provide valve to drain pond for maintenance.
Pollutant Removal	Moderate to high.

12.13 INFILTRATION CONTROLS

12.13.1 <u>Introduction</u>

Infiltration controls are best management practices (BMPs) where the primary discharge of stormwater is to the groundwater table. These include infiltration trenches, infiltration basins and porous pavement. In some cases, the stormwater is intercepted after it has infiltrated a few feet n underdrain and is discharged to a storm sewer or surface water. One of the primary concerns with the use of infiltration BMPs is the risk of groundwater contamination. This is why there should be at least 5 ft between the bottom of the facility to the seasonable high-water table and 5 ft to the underlying bedrock. Another factor is the residence time in the facility. Sources recommend that the first-flush stormwater be infiltrated within 24 h to 72 h. The infiltration rate is directly related to the soil type and disposition. A soil investigation should be performed at all facility locations prior to construction. Table 12-10 provides some considerations in evaluating an infiltration control.

TABLE 12-10 — Summary of Considerations for an Infiltration Facility

Quality	Infiltrate WQV within 48 h; minimum residence time of 24 h.
Quantity	Control 2- and 10- yr peak flows (could lead to a large expensive facility; could be used with detention pond to control quantity).
Shape	Dependent on site constraints.
Maintenance	Inspect once a year; preferably during wet weather; mow area twice a year; remove sediment every 5 to 10 years.
Other Considerations	Filter strip to remove sediments 2% to 5% slope with minimum 20 ft length; infiltration rate minimum 1 in/h; depth to groundwater and bedrock 5 ft; effects of facility on quality of groundwater.
Pollutant Removal	Moderate to high.

12.13.2 Site Selection

To evaluate different sites, a report from the Maryland State Highway Administration (MDSHA) *(9)* described a procedure that rates different sites by using several parameters (reader is referred to this study for details of the procedure). Upon completion of this procedure, several sites can be compared to determine which is the best for the infiltration BMP or if infiltration is even feasible. Other selection considerations include size of drainage area and proximity to foundations (the facility should be no closer than 10 ft down-gradient and 100 ft up-gradient from a foundation).

12.13.2.1 Infiltration Rate

The infiltration rate is rarely used to determine the outflow from the facility for quantity and quality control. Infiltration rates of greater than 1 in/h are preferred for infiltration facilities. After a suitable site for the facility has been found, several soil tests must be made before the facility is designed.

First, borings should be dug at the site to determine the soil types, depth to bedrock and groundwater and infiltration rates. These parameters can also be determined from county soil maps. The infiltration rate can be determined with varying elevation heads. This can be done with the “falling-head test” in the field. The test was described in Reference *(9)*. This procedure will yield a curve of outflow versus storage, which can be used to route storms through the facility.

12.13.2.2 Observation Well

An observation well should be included in an infiltration facility with a covered bottom (i.e., trenches and pavement) to allow an inspector to determine how well the facility is operating (e.g., whether the stormwater is infiltrating as designed or whether maintenance is required). A schematic of a typical observation well in an infiltration trench is shown in Figure 12-14. It may also be necessary to install wells in infiltration basins to determine if they are working properly, but this can be determined visually because the stormwater is stored on the surface whereas the storage in trenches and pavement is hidden from view.

12.13.3 Infiltration Trench

An infiltration trench (see Figure 12-15) is a facility where a trench is excavated and then filled with a porous medium. Stormwater is stored in the voids of the fill material until it can be infiltrated. In a variation of this design, the stormwater is collected by an underdrain pipe after the stormwater has been detained and filtered by the trench. Infiltration trenches can be used in median strips or adjacent to parking lots.

12.13.3.1 Quality

The bottom of the facility must be 5 ft above the bedrock and the seasonably high groundwater table, and the bottom of the infiltration trench must be below the frost line. The WQV must be infiltrated within 48 h. The primary removal mechanisms in trenches are sedimentation and filtration, along with some biological uptake. Filtering is achieved in the top layers of the facility as stormwater enters. In the infiltration trench, the main removal mechanisms are sedimentation and adsorption.

As the stormwater leaves, it is filtered again by the underlying soil, where more pollutants will be removed. Unfortunately, all infiltration facilities are vulnerable to clogging, thereby reducing their effectiveness. Therefore, a vegetated buffer strip filtering the runoff is recommended as part of an infiltration facility. The strip would decrease the amount of suspended solids in the stormwater and thus increase the useful life of the infiltration facility. The filter strip should be at least 20 ft wide. It should also be sloped from 2% to 5% to prevent water from ponding and to ensure a slow velocity.

12.13.3.2 Quantity

Because of the large size of the trench that would be required to control a 10-yr storm, it is suggested that trenches not be used for large drainage areas or areas where the increase in peak flow and, therefore the amount of storage required, is very large.

A trial-and-error process of routing the design storms through the facility can be used to determine the amount of storage required for quantity control. Although the methods used to determine the amount of storage required for the ponds are not derived for infiltration BMPs, they can still be used to obtain an initial estimate of the required storage.

After a storage volume has been determined, the dimensions of the facility can be estimated. The depth should be designed such that the bottom is 5 ft above the bedrock and high-water table and below the frost line. The surface area can be manipulated to suit the site conditions if it yields the required storage volume. The amount of surface area required is:

$$S_a = \frac{Vol_s}{V_r d} \tag{12.17}$$

where:
- S_a = surface area, ft^2
- Vol_s = storage volume, ft^3
- V_r = void ratio (= 0.4 for 1½ in to 3 in aggregate)
- d = depth, ft

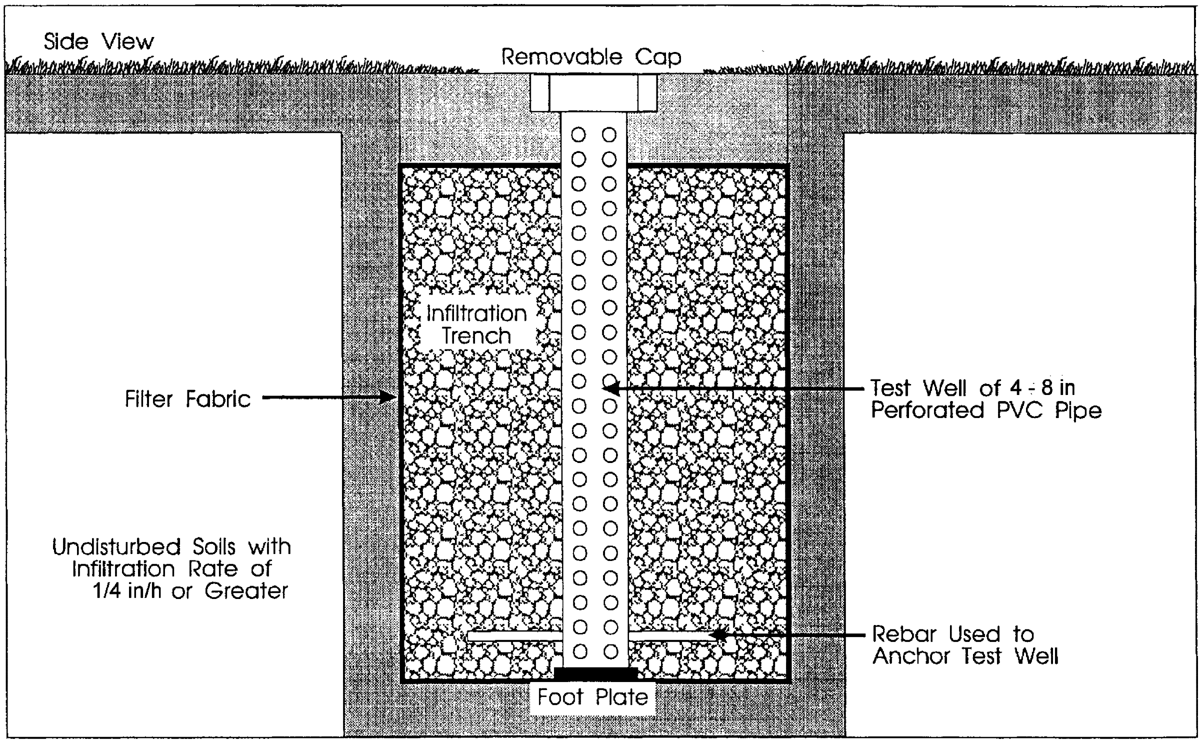


FIGURE 12-14 — Infiltration Trench With Observation Well (after Reference *(11)*)

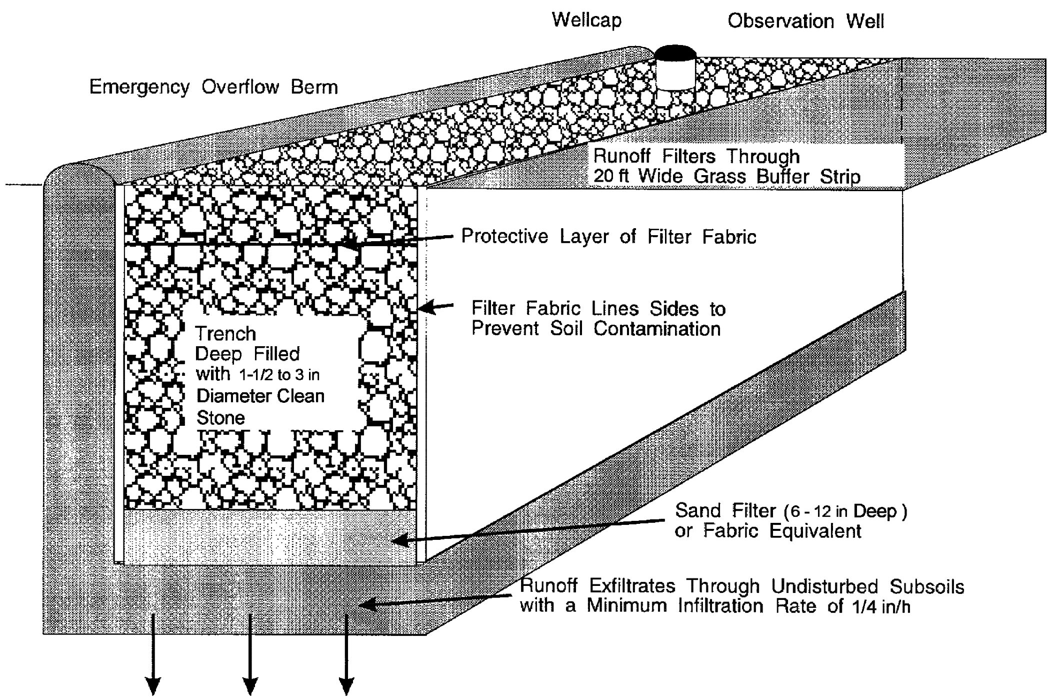


FIGURE 12-15 — Infiltration Trench (after Reference *(11)*)

The inflow hydrograph can be calculated by a number of methods, and the outflow-versus-storage curve can be found from a falling-head test. Thus, the storms can be routed through the facility, and the size of the trench can be changed to reduce the peak outflows to the preconstruction levels of a 2- and 10-yr design storm. When routing the storm through the trench, one can determine whether and how much flow will bypass the trench when it is filled with stormwater. The overflow from the trench must be contained in an adequate channel.

12.13.3.3 Other Considerations

Detention time is an important factor in determining the effectiveness of a trench facility. A facility which drains quickly is capable of treating more stormwater volume. A maximum detention time of 72 h is recommended. The actual detention time can be estimated by:

$$T_s = \frac{dV_r}{f} \tag{12.18}$$

where:
- T_s = storage time or detention time, h
- d = depth of storage in the trench, in
- V_r = void ratio of stone reservoir
- f = steady infiltration rate, in/h

From this Equation, it can be seen that detention time is directly related to trench depth. Because the WQV will most likely be much smaller than the storage required for a 10-yr storm, the depth of the WQV will be very small in the trench. Therefore, infiltration trenches are much better suited for small drainage areas where the change in peak flow between pre- and post-construction is small. Modifications can be made to the trench design to increase the depth of the WQV storage, but these will increase the cost and could make this BMP option infeasible.

12.13.3.4 Quality

To protect groundwater quality, the bottom of the infiltration basin must be 5 ft or more above the bedrock and the seasonably high groundwater table. The WQV should be infiltrated within 48 h. The primary removal mechanisms in infiltration basins are sedimentation, filtration and biological uptake. Filtering is provided by the vegetation at the bottom of the pond and, preferably, also by a buffer strip before the stormwater runoff enters the facility. The filter strip should be at least 20 ft wide and should also be sloped from 2% to 5% to prevent water from ponding and to ensure a slow velocity. Vegetation can also contribute to the removal of pollutants through biological uptake. As the stormwater leaves, it is filtered again by the underlying soil.

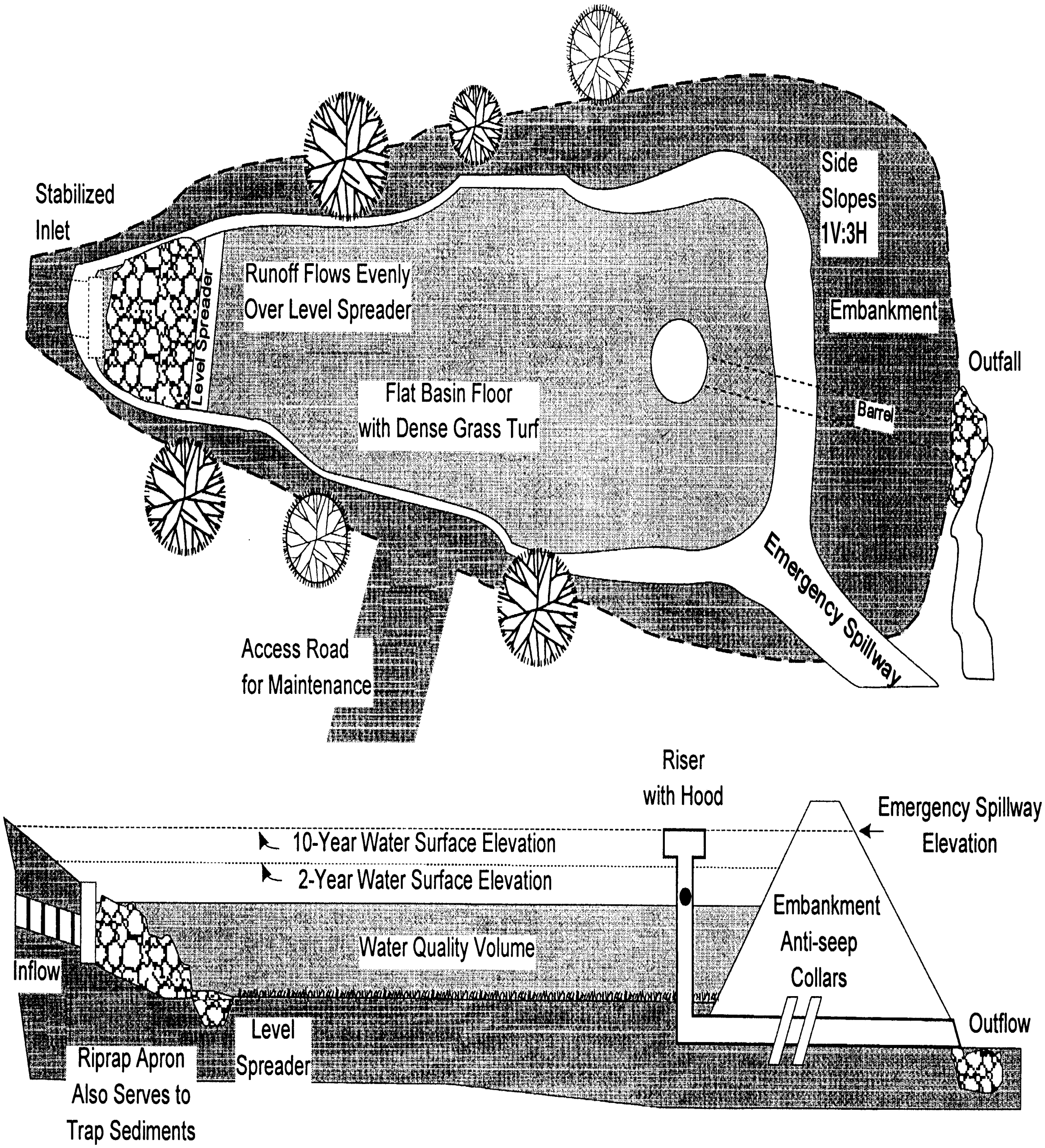


FIGURE 12-16 — Infiltration Basin (after Reference *(11)*)

An estimation of the maximum ponding depth for a desired drain time can be found with the equation:

$$d = fT_s \tag{12.19}$$

where: d = depth, in
f = steady infiltration rate, in/hr

T_s = time of storage, h

The recommended maximum allowable storage time is 48 h. Considering that basins may fail because of clogging and an infiltration rate that is lower than expected, a shorter time of storage, say 40 h, might be used to compensate for inaccuracies in estimating infiltration rates.

Several other considerations can enhance the pollutant removal of these facilities. First, vegetation should be established on the basin floor. A dense stand of water-tolerant grass with a deeply penetrating root system would help stabilize the bottom of the basin and help keep the soil open. Vegetation would also provide the biological uptake of nutrients.

Second, the pond bottom should be sloped as close to zero as possible to obtain a uniform depth of stormwater over the basin. The side slopes should be sloped at 1V:3H, or flatter, to allow for easy maintenance access and prevent erosion.

A third consideration pertains to the incoming stormwater. A combination of a level spreader/sediment forebay can be constructed to spread the stormwater evenly, thereby reducing erosion, and trap sediments before they clog the basin. Riprap should also be placed at the inlet to help reduce erosion.

12.13.3.5 Quantity

For an infiltration basin, quantity can be controlled very similarly to a detention basin. The basin should be designed to reduce the peak flow from a 2-yr and a 10-yr storm, considered individually and be able to pass a 100-yr storm safely.

After a required storage volume is estimated, the design storms should be routed through the basin to determine if the estimated value is correct.

12.13.4 Porous Pavement

Porous pavement is an infiltration practice in which a stone “reservoir” is placed under a layer of open-graded asphalt pavement course that contains no fines, yielding a pavement with approximately 16% voids that allows water to infiltrate. Under the asphalt is a stone reservoir that stores stormwater. This type of facility is not to be confused with an open-graded bituminous concrete surface course used to reduce water filming on highway surfaces. Figure 12-17 shows a cross section of a typical design. Porous pavement is generally not recommended for highway uses but is more appropriate for parking lots and other low-traffic areas. There have also been structural problems and clogging problems in some applications. Because the stone reservoir is located under the asphalt layer, maintenance can be difficult and costly. Winter salts and other abrasives should not be applied to the facility because they may cause clogging. Vacuuming on a regular basis is recommended.

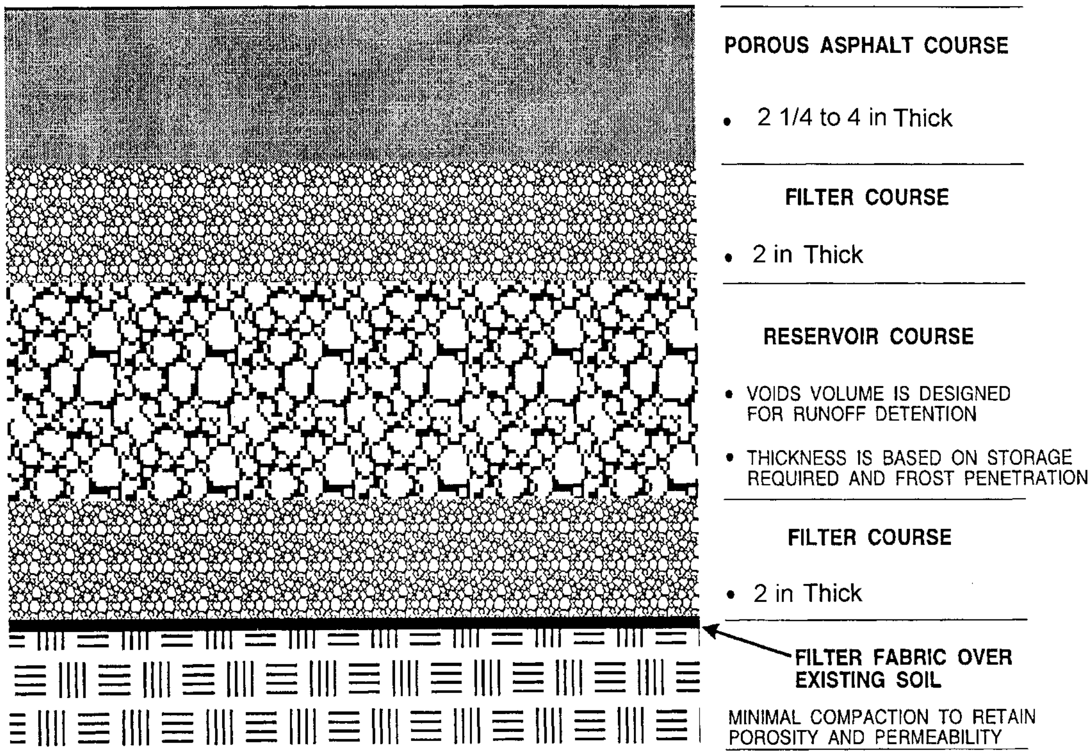


FIGURE 12-17 — Porous Pavement
(After Reference *(5)*)

12.13.4.1 Quality

For quality, porous pavement is designed similarly to an infiltration trench. The WQV must be infiltrated within 48 h, and the bottom of the facility must be 5 ft above the seasonably high-water table and bedrock and below the frost line. Surface area is usually controlled by the size of the parking lot, and depth is controlled by the time of storage and the infiltration rate.

The following equations can be used to determine the required depth and surface areas (S_A), respectively:

$$d = T_s f / V_r \qquad (12.20)$$

$$S_A = WQV / V_r d \qquad (12.21)$$

where:
- d = depth, in
- T_s = storage time, h
- f = steady infiltration rate, in/hr
- V_r = void ratio
- WQV = water quality volume, ft^3

12.13.4.2 Quantity

For quantity, the peak flows from a 2- and 10-yr storm must be reduced to predevelopment levels. The reduction can be accomplished by using the storage reservoir under the pavement to store the increased runoff from the developed area.

After a storage volume has been estimated, the design storms should be routed through the facility to determine if the estimated storage volume is adequate. Adjustments should be made if necessary.

Some designs may include an outlet pipe from the stone reservoir to drain excess stormwater and, therefore, provide control of the larger storms.

12.13.5 Vegetative Control

Vegetative controls can be used to reduce the size and cost of structural water quality controls. Their use in conjunction with structural controls to reduce the size of a project and improve its controlling of runoff quantity and quality is encouraged. Often, vegetative systems are the only feasible effective runoff treatment measure available for use prior to release of stormwater to a wetland system.

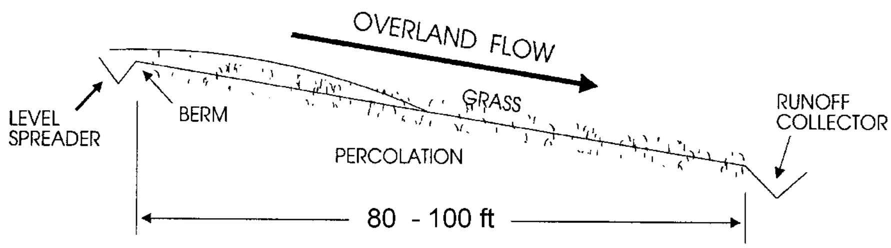
12.13.5.1 Filter Strip

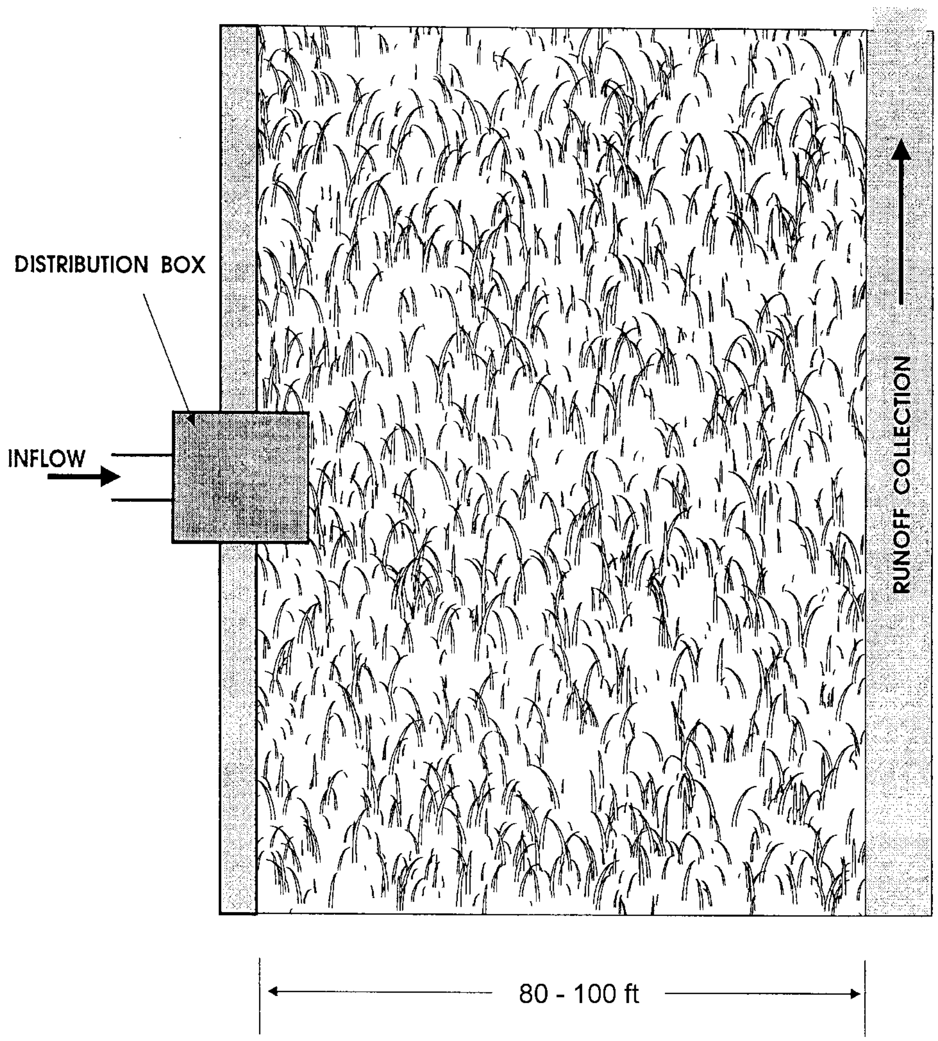
A filter strip is a vegetated area that is designed to accept sheet flow. While flowing over the strip, stormwater is filtered by the vegetation, infiltrated and detained. The most common cause for failure of filter strips is runoff bypassing the strip through eroded channels. If the stormwater is not evenly distributed over the entire strip, a channel could form and the strip would lose effectiveness. To prevent the channelization, a level spreader can be used, as shown in Figure 12-18.

Filter strips can be used to filter runoff before it enters a structural facility, or they could be used alone. A study by Yu, et al. *(19)* found that the level spreader was at least as cost effective as a wet pond for pollutant removal in stormwater. However, its use for quantity control is limited to small drainage areas, with small increases in peak flows.

Filter strips should be constructed of dense, soil-binding, deep-rooted, water-resistant plants. They are usually constructed of grass, but forested strips are also feasible (they can have higher pollutant removal rates but should be longer because of their lack of cover and susceptibility to erosion). For the filter strips to be effective, their slope should be no more than 5%, and their length should be at least 20 ft.

Figure 12-19 was developed by Wong and McCuen *(16)* for determining the required length of a grassed filter strip. If the slope of the strip, roughness coefficient (Manning's n) and desired trap efficiency are known, the length required can be found by using Figure 12-19. The example in Figure 12-19 is for a slope of 2%, an n of 0.20 and a trap efficiency of 95%; the required filter strip length is 200 ft.


OVERLAND FLOW
GRASS
LEVEL SPREADER
BERM
PERCOLATION
RUNOFF COLLECTOR
80 - 100 ft


DISTRIBUTION BOX
INFLOW
RUNOFF COLLECTION
80 - 100 ft

PLAN VIEW

FIGURE 12-18 — Level Spreader System

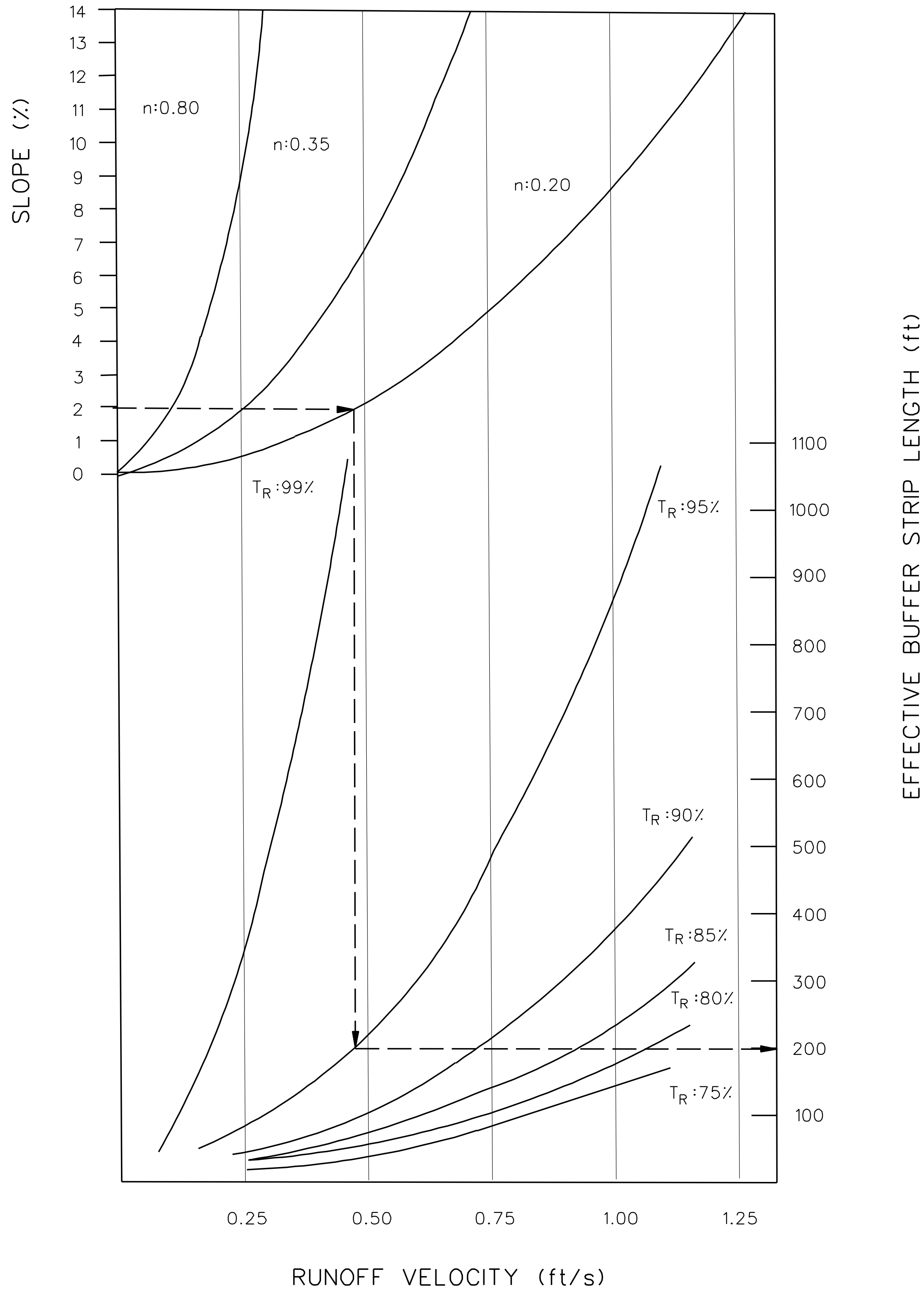


FIGURE 12-19 — Removal Rates (R_R) For Buffer Strips (after Reference *(16)*)

As previously stated, the use of a level spreader is intended to spread runoff evenly and prevent the formation of channels in the filter strip. Several designs have been developed; the main consideration is that the overflow from the level spreader be distributed equally across the filter strip. This can be done through the use of a rock-filled trench or a plastic-lined trench that acts as a small detention pond. The bottom and filter-side lip should have a zero slope to ensure an even distribution of runoff onto the strip. Figure 12-18 depicts a level spreader.

12.13.5.2 Grassed Swale

Grassed swales are roadside stormwater conveyances that can store, filter and infiltrate runoff. Originally, they were an inexpensive way of rapidly transporting runoff from a site. In constrast, runoff should be slowed down and detained for SWM purposes.

Some studies have been conducted on the use of swales for runoff quality control, and a wide variety of estimates of their effectiveness have been reported. From these studies, design guidelines have been developed for constructing swales so that the pollutant removal efficiency is improved.

DESIGN. Several studies conducted by Yousef, et al *(18)* for the Florida Department of Transportation found roadside swales to be effective in removing many highway pollutants. An equation was developed to calculate the length of a swale that allows all of the stormwater to infiltrate for a given runoff flow rate:

$$L = \frac{KQ^{5/8}S^{3/16}}{n^{3/8}f} \qquad (12.22)$$

where:
- L = length of swale, ft
- K = constant (see Table 12-11)
- Q = average runoff flow rate, ft^3/s
- S = longitudinal slope, ft/ft
- n = Manning’s roughness coefficient
- f = infiltration rate, in/hr

If this leads to a swale that is too long, another equation may be used to determine the placement of swale blocks or check dams to compensate for the reduction in length where required because of site limitation, etc. By modifying the equation, the volume of storage required in the swale can be determined by:

TABLE 12-11 — Swale Length Constant, K

Z (Side Slope) (1V:ZH)	K	Z (Side Slope) (1V:ZH)	K
1	981 000	6	485 000
2	854 000	7	443 000
3	712 000	8	408 500
4	612 000	9	380 000
5	540 000	10	357 600

Source: Reference *(15)*.

$$\mathrm{Vol} = Q(\Delta t) - \left(\frac{Ln^{3/8}f}{KS^{3/16}}\right)^{8/5}(\Delta t) \qquad (12.23)$$

where: $Q(\Delta t)$ = volume of runoff at the end of time interval Δt, s

Δt = time interval, s

$\left(\frac{Ln^{3/8}f}{KS^{3/16}}\right)^{8/5}$ = the value of runoff percolated during time interval Δt by a certain swale with length L

Units of all other variables are the same as defined in Equation 12.22. The answer obtained by solving Equation 12.23 is, therefore, the volume that must be stored behind the check dam in the swale so that all stormwater is allowed to infiltrate.

The pollutant removal efficiency of a swale can be improved through enhancing filtering by grass in the channel. To enhance grass filtering, the swale should be designed as a triangle, with at least 1V:3H side slopes, or a parabola, with a 6:1 top width-to-depth ratio. The grass in the swale should be dense, deep rooted and water tolerant. The grass should be high enough to cover the depth of runoff in the swale but not so high that it is flattened by the flowing stormwater.

12.13.6 Wetlands

Wetlands have the ability to remove many pollutants, and wetlands detain stormwater. However, the processes that occur in wetlands are not fully understood, and the amount of wetland area required to treat stormwater can be very large. It has been recommended that wetlands and marshes be used in conjunction with other BMPs, such as on the bottom of dry ponds and on the fringes of wet ponds. Although a substantial amount of information is available on using wetlands as a final treatment process of wastewater, very little is known on using wetlands for treating stormwater. A report by Marble *(7)* provided guidelines for designing replacement wetlands. With regard to using wetlands for SWM, Marble reported that urban runoff is a good source of nutrients for the development of wetlands and that wetlands downstream of an impoundment may have reduced aquatic diversity because of reductions in the outflow detritus. Marble further stated that wetlands have the ability to remove sediments and toxins through sedimentation. However, the loadings of toxins and sediments should be low to moderate, and the ratio of wetland area to watershed area should be kept high. The functions of wetlands with regard to water quality are very complex. Hemond and Benoit *(6)* noted the following:

> *The wetland is not a simple filter; it embodies chemical, physical and biotic processes that can detain, transform, release, or produce a wide variety of substances. Because wetland water quality functions result from the operation of many individual, distinct and quite dissimilar mechanisms, it is necessary to consider the nature of each individual process.*

The very limited number of studies undertaken on the use of wetlands for SWMs indicate a wide disparity in the efficiency of wetlands to remove pollutants. A study by Martin *(8)* suggested that wetlands, when used in conjunction with another BMP (e.g., a wet detention pond), can be quite effective in treating highway stormwater runoff. Because State DOTs are required to replace wetlands on a routine basis, the idea of using these constructed wetlands for SWMs appears to be a prudent one. However, more field test and monitoring data need to be collected and analyzed before appropriate design guidelines can be developed.

12.14 LAND-LOCKED RETENTION

12.14.1 Introduction

Watershed areas that drain to a central depression with no positive outlet (playa lakes) are typical of many topographic areas including karst topography and can be evaluated using a mass-flow routing procedure to estimate flood elevations. Although this procedure is fairly straightforward, the evaluation of basin outflow is a complex hydrogeologic phenomenon that requires good field measurements and a thorough understanding of local conditions. Because outflow rates for flooded conditions are difficult to calculate, field measurements are desirable.

12.14.2 Mass Routing

The Steps presented below for the mass-routing procedure are illustrated by the example given in Figure 12-20:

Step 1 Obtain cumulative rainfall data for the 100-yr frequency, 6-d duration design event from Figure 12-21.

Step 2 Calculate the cumulative inflow to the land-locked retention basin using the rainfall data from Step 1 and runoff procedure from the Hydrology Chapter. Plot the mass inflow to the retention basin.

Step 3 Develop the basin outflow from field measurements of hydraulic conductivity, considering worst-case watertable conditions. Hydraulic conductivity should be established using in-situ test methods, then results compared to observed performance characteristics of the site. Plot the mass outflow as a straight line with a slope corresponding to worst-case outflow in in/h.

Step 4 Draw a line tangent to the mass-inflow curve from Step 2, which has a slope parallel to the mass-outflow line from Step 3.

Step 5 Locate the point of tangency between the mass-inflow curve of Step 2 and the tangent line drawn for Step 4. The distance from this point of tangency and the mass-outflow line represents the maximum storage required for the design runoff.

Step 6 Determine the flood elevation associated with the maximum storage volume determined in Step 5. Use this flood elevation to evaluate flood protection requirements of the project. The zero-volume elevation shall be established as the normal wet season water surface or watertable elevation or the pit bottom, whichever is highest.

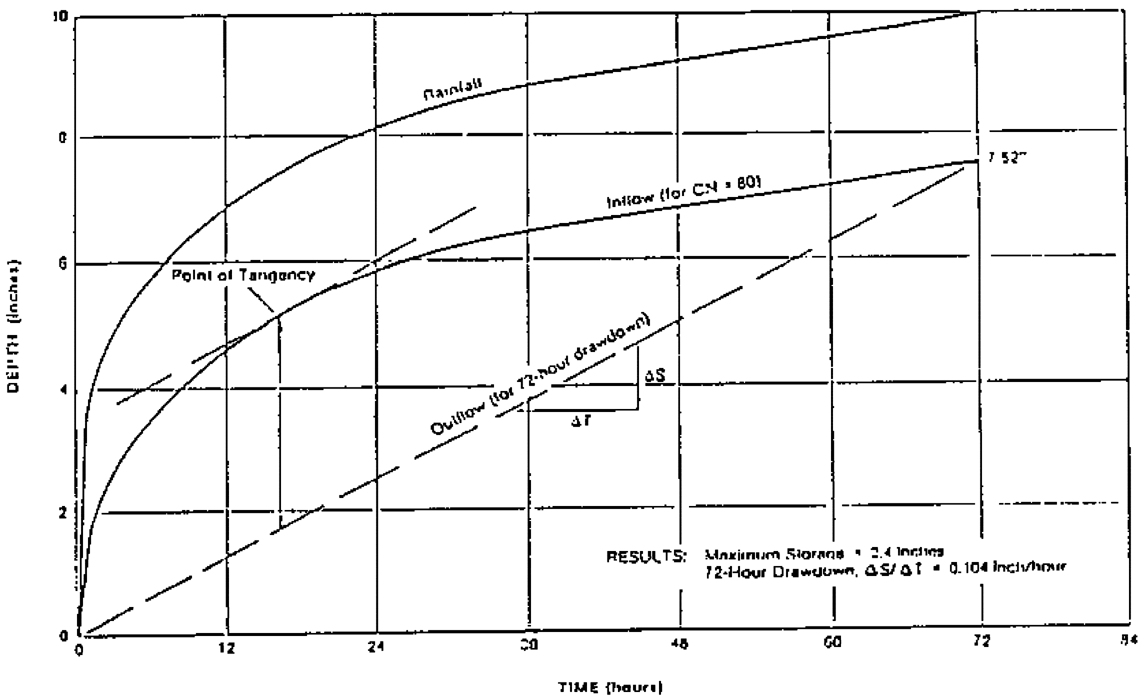


FIGURE 12-20 — Mass Routing Curve

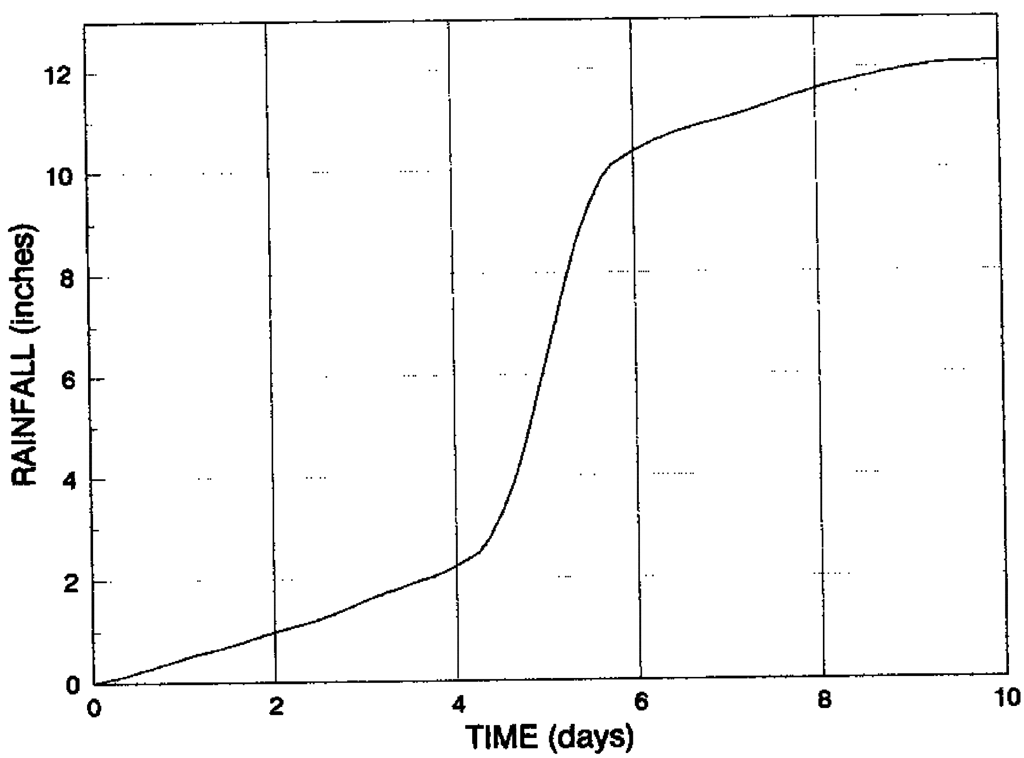


FIGURE 12-21 — Cumulative Rainfall Data for 100-Yr, 10-D Design Storm
((Agency insert local graph))

Step 7 If runoff from the project area discharges into a drainage system tributary to the land-locked depression, detention storage facilities are required to comply with the pre-development discharge requirements for the project.

Unless the storage facility is designed as a retention facility, including water budget calculations, environmental needs and provisions for preventing anaerobic conditions, relief structures shall be provided to prevent standing water conditions.

12.15 RETENTION STORAGE FACILITIES

12.15.1 Introduction

The use of retention storage facilities that have a permanent pool (wet ponds) is often discouraged because of the extensive maintenance that is sometimes required. Provisions for weed control and aeration for prevention of anaerobic conditions ((shall)) be considered. Also, facilities should not be built that have the potential for becoming nuisances or health hazards. Note that wet ponds are required where water quality problems are to be addressed.

12.15.2 Water Budget

Water budget calculations are required for all permanent pool facilities and should consider performance for average annual conditions. The water budget should consider all significant inflows and outflows including, but not limited to, rainfall, runoff, infiltration, exfiltration, evaporation and outflow.

Average annual runoff may be computed using a weighted runoff coefficient for the tributary drainage area multiplied by the average annual rainfall volume. Infiltration and exfiltration should be based on site-specific soils testing data. Evapotranspiration can be calculated by using the Blanay-Criddle method.

12.16 EXAMPLE PROBLEM

12.16.1 Example

A shallow basin with an average surface area of 3 ac and a bottom area of 2 ac is planned for construction at the outlet of a 100-ac watershed. The watershed is estimated to have a post-development runoff coefficient of 0.3. Site-specific soils testing indicates that the average infiltration rate is approximately 0.1 in/h. Determine for average annual conditions if the facility will function as a retention facility with a permanent pool.

12.16.2 Solution

1. From rainfall records, the average annual rainfall is approximately 50 in.

2. The mean annual evaporation is 35 in.

3. The average annual runoff is estimated as:

   Runoff = (0.3)(50 in)(100 ac) = 1,500 ac•in

4. The average annual evaporation is estimated as:

   Evaporation = (35 in)(3 ac) = 105 ac•in

5. The average annual infiltration is estimated as:

   Infiltration = (0.1 in/hr)(24 h/d)(365 d/yr)(2 ac)
   Infiltration = 1,752 ac•in

6. Neglecting basin outflow and assuming no change in storage, the runoff (or inflow) less evaporation and infiltration losses is:

   Net Budget = 1,500 – 105 – 1,752 = -357 ac•in

   Thus, the proposed facility will not function as a retention facility with a permanent pool.

7. Revise pool design as follows:

   Average surface area = 2 ac and bottom area = 1 ac

8. Recompute the evaporation and infiltration:

   Evaporation = (35 in)(2 ac) = 7 ac•ft
   Infiltration = (0.1 in/hr)(24 h/d)(365 d/yr)(1 ac) = 876 ac•in

9. The revised runoff less evaporation and infiltration losses is:

   Net Budget = 1,500 – 70 – 876 = +554 ac•in

The revised facility is assumed to function as a retention facility with a permanent pool.

12.17 CONSTRUCTION AND MAINTENANCE CONSIDERATIONS

12.17.1 General

An important step in the design process is identifying whether special provisions are warranted to properly construct or maintain proposed storage facilities. To assure acceptable performance and function, storage facilities that require extensive maintenance are discouraged. The following maintenance problems are typical with urban detention facilities, and facilities ((shall)) be designed to minimize problems:

- weed growth,
- grass and vegetation maintenance,
- sedimentation control,
- bank deterioration,
- standing water or soggy surfaces,
- mosquito control,
- blockage of outlet structures,

- litter accumulation, and
- maintenance of fences and perimeter plantings.

Proper design should focus on the elimination or reduction of maintenance requirements by addressing the potential for problems to develop:

- Both weed growth and grass maintenance may be addressed by constructing side slopes that can be maintained using available power-driven equipment (e.g., tractor mowers).

- Sedimentation may be controlled by constructing traps to contain sediment for easy removal or low-flow channels to reduce erosion and sediment transport.

- Bank deterioration can be controlled with protective lining or by limiting bank slopes.

- Standing water or soggy surfaces may be eliminated by sloping basin bottoms toward the outlet, constructing low-flow pilot channels across basin bottoms from the inlet to the outlet, or constructing underdrain facilities to lower watertables.

- In general, when the above problems are addressed, mosquito control will not be a major problem.

- Outlet structures should be selected to minimize the possibility of blockage (i.e., very small pipes tend to block quite easily and should be avoided).

- Finally, one way to address the maintenance associated with litter and damage to fences and perimeter plantings is to locate the facility for easy access where maintenance can be conducted on a regular basis.

12.17.2 Sediment Basins

Often detention facilities are used as temporary sediment basins. To control the maintenance of these facilities, some criteria must be established to determine when these facilities should be cleaned and how much of the available storage can be used for sediment storage.

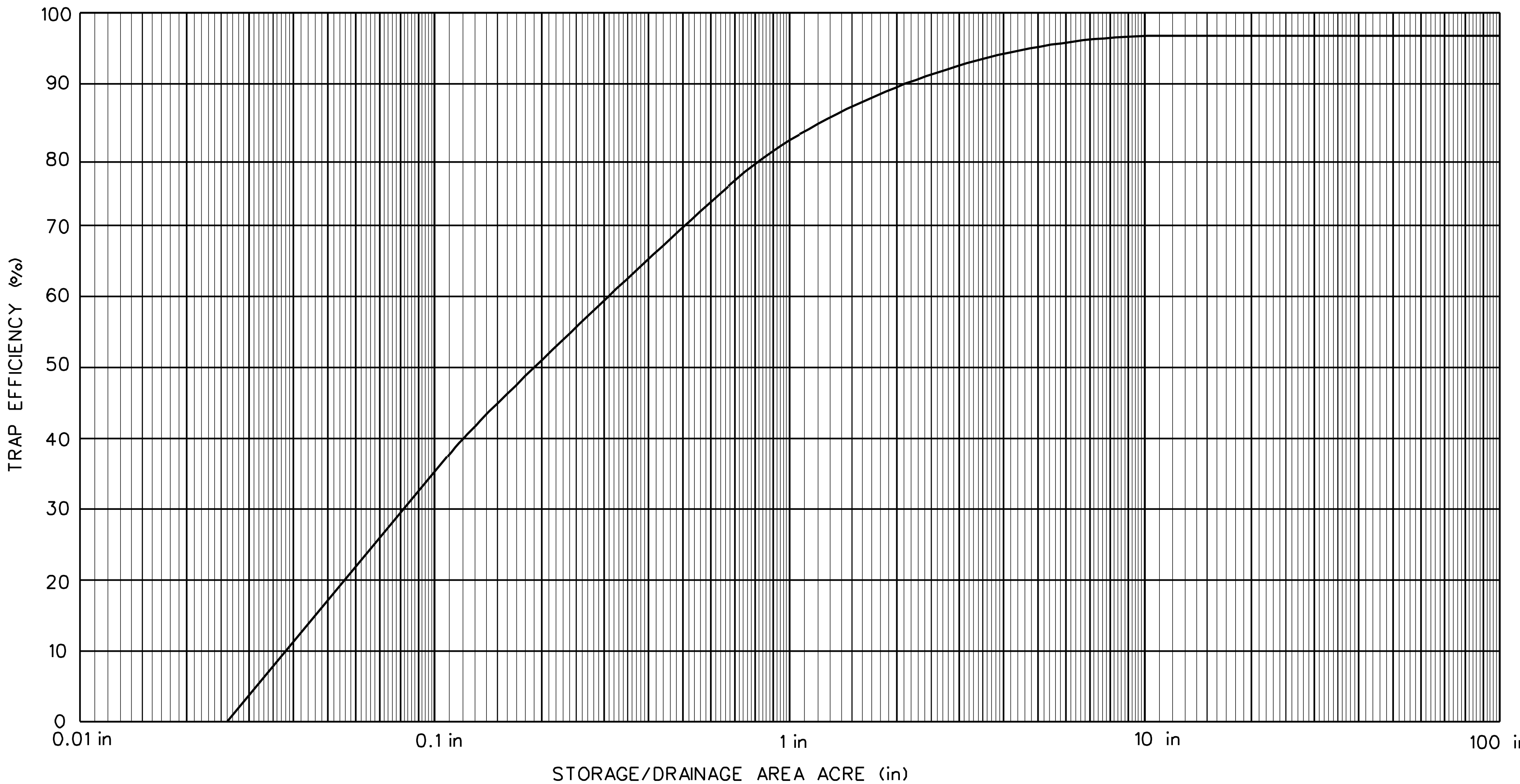
The following is an example methodology that could be used to develop a sediment basin maintenance schedule. Figure 12-22 can be used to estimate sediment-trap efficiency for sediment basins with different volumes. The procedure for using this Figure is as follows:

1. Establish sediment-generation criteria (e.g., 1786 ft^3 silt per disturbed acre draining to the sediment basin).

2. Estimate total volume available for sediment storage from the geometric shape of the basin (e.g., 19,421 ft^3).

3. Calculate minimum silt storage needed given the silt generation criteria (e.g., 1,786 ft^3 per disturbed acre $\times$ 9.88 ac of disturbed area = 17,646 ft^3).

4. Trap efficiency can be estimated from Figure 12-22 as follows:



Note: Capacity is total sediment basin volume up to emergency spillway crest. From *(2)*.

FIGURE 12-22 — Efficiency of Sediment Basins

- 19,421 ft^3 available storage/((9.88 ac)(1 ac/43,560 ft^2)) = 0.045 ft
- (0.045 ft)(12 in/ft) = 0.54 in (storage/drainage area (acre))
- from Figure 12-22 at 0.54 in, Trap Efficiency = 72%

5. If one is required to clean out the facility when the efficiency reaches 50%, the clean-out elevation could be determined as follows:

    - From Figure 12-22 at 50% trap efficiency, the storage/drainage area = 0.185 in.
    - Storage = (0.185 in)(9.88 ac)(1 ft/12 in)(43,560 ft^2/1 ac) = 6635 ft^3.
    - Given the basin geometry, try different depths until one has 6635 ft^3 of storage still available for sediment storage. This is then the depth where the basin should be cleaned to ensure that the trap efficiency does not fall below 50%.

This illustrates a procedure that can be used to estimate trap efficiency and also establish clean-out levels for a given efficiency. Different efficiencies might be established according to the damage potential downstream.

12.18 PROTECTIVE TREATMENT

Protective treatment may be required to prevent entry to facilities that present a hazard to children and, to a lesser extent, all persons. Fences may be required for detention areas where one or more of the following conditions exist:

- Rapid-stage increases would make escape practically impossible where small children frequent the area.
- Water depths either exceed 3 ft for more than 24 h or are permanently wet and have side slopes steeper than 1V:4H.
- A low-flow watercourse or ditch passing through the detention area has a depth greater than 5 ft or a flow velocity greater than 5 ft/s.
- Side slopes equal or exceed 1V:1.5H.

Guards or grates may be appropriate for other conditions but, in all circumstances, heavy debris must be transported through the detention area. In some cases, it may be advisable to fence the watercourse or ditch rather than the detention area.

Fencing should be considered for dry retention areas with design depths in excess of 3 ft for 24 h, unless the area is within a fenced, limited-access facility.

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